# COUNTY HEIGHTS DRAINAGE BASIN

# DESIGN PLAN AMENDMENT

## CITY OF RAPID CITY
**PROJECT #DR10-1870**
**CIP 50757**

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SEPARATE COVER – DVD CONTAINING:
HECHMS Existing Conditions Files (Version 3.5)
HECHMS DBDPA Conditions Files (Version 3.5)
REPORT in PDF and WORD
EXCEL and PDF Data Table Spreadsheets
AUTOCADD Files

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## ABBREVIATIONS

| | |
|---|---|
| CMP | Corrugated Metal Pipe |
| CN | NRCS Curve Number |
| CUHP | Colorado Urban Hydrograph Procedure Computer Model |
| DBDP | Drainage Basin Design Plan |
| DBDPA | Drainage Basin Design Plan Amendment |
| EPA | Environmental Protection Agency |
| FEMA | Federal Emergency Management Agency |
| FLOWMASTER | Computer Model Flowmaster, Bentley Systems Inc. |
| FMG | FMG, Inc, Consulting Engineers, Rapid City, SD |
| HMS | HECHMS Computer Modeling Program, US Army |
| HY8 | HY8 Computer Model, Federal Highway Administration |
| LID | Low Impact Development |
| MS4 | Small Municipal Separate Storm Sewer System |
| NRCS | Natural Resources Conservation Service (formerly SCS) |
| NWI | National Wetlands Inventory |
| RCIDCM | City of Rapid City Infrastructure Design Criteria Manual |
| RCP | Reinforced Concrete Pipe |
| SDDENR | South Dakota Department of Environment and Natural Resources |
| SDDOT | South Dakota Department of Transportation |
| TRM | Turf Reinforcement Mat |
| UDSWM | Urban Drainage Storm Water Management Model |
| USCOE | United States Army Corps of Engineers |

# 1. INTRODUCTION

## 1.0 BACKGROUND AND STUDY AREA

This Drainage Basin Design Plan Amendment has been prepared by FMG Inc., for the City of Rapid City under Project Number DR10-1870/CIP Number 50757.

The County Heights Drainage Basin is approximately 1,702 acres. The County Heights Drainage Basin is shown on Figure 1. Further discussion related to the study area is found in Section 3 of this report.

## 1.1 PURPOSES AND OBJECTIVES

The purposes and objectives of this DBDPA are:

- Provide an update to the original County Heights Drainage Basin Design Plan to (1) account for current hydraulic improvements, including detention ponds and major conveyance features, (2) account for current and proposed land use types in the basin, many of which differ from the original DBDP assumptions, and (3) account for new design criteria per the City of Rapid City Infrastructure Design Criteria Manual.
- Provide HMS computer models of the basin to replace the current CUHP/UDSWM models. HMS V3.5 was used for this project.
- Provide computer models and output for 2 year, 10 year, and 100 year storms.
- Provide conceptual design guidance for future improvements.
- Provide general recommendations for storm water quality treatment.
- Provide other information as necessary to adequately describe the needs of the DBDPA.

## 1.3 REPORT ORGANIZATION AND DELIVERABLES

This DBDPA is organized in the following major sections:

1. Introduction
2. Supporting Literature and Data
3. General Basin Information
4. Wetlands
5. Sub-basin Hydrology
6. Hydraulics Overview
7. Channel and Pipe Elements
8. Detention Ponds
9. Junction Elements
10. Minor Sub-Basins
11. Irrigation Ditches
12. Storm Water Quality
13. Major Recommendations Summary, Cost Estimate, and Prioritization
14. Modeling Comparisons

Appendix A Data & Printouts for Existing Land Use & Existing Hydraulic Conditions
Appendix B Data & Printouts for Future Land Use & Future (DBDPA) Hydraulic Conditions
Appendix C Hydrographs for Future Land Use & Future (DBDPA) Hydraulic Conditions
Appendix D Photographs

The full deliverable package for this DBDPA includes:

- Bound Copy of DBDPA dated January 18, 2016
- HEC-HMS Existing and DBDPA models (on DVD)
- Other digital data pertinent to study
- PDF of DBDPA (on DVD)

## 1.4 STUDY LIMITATIONS

It was beyond the scope of work to provide engineering design and drawings suitable for construction. The DBDPA presented herein is conceptual in nature and is intended to provide the general information necessary for the final design of a fully planned major drainage system.

It is unlikely that the final design of any improvement will exactly follow the recommendations presented in this report; therefore, it will be necessary to make a final detailed technical analysis of any of the proposed improvements prior to their construction. The final detailed technical analysis must include computer analysis of the entire system to insure the proposed improvements do not have a negative impact elsewhere by changing of runoff conditions, lag times, etc. An official Amendment should be prepared and documented by the appropriate agencies if warranted by changes.

All users of this DBDPA should check with the City of Rapid City to determine if this document has been further amended.

This DBDPA provides for only major drainage. Unless specifically noted in the study, localized or minor drainage was beyond the scope of work. It is also noted the recommendations for major drainages include only significant issues and such things are minor erosion, riprap displacement, maintenance issues, etc., have are not included unless specifically discussed.

Unless otherwise noted, it was beyond the scope of the project to perform field surveys of any features. Data used in this study was obtained from City of Rapid City GIS aerial photography, City of Rapid City GIS aerial contours, as built construction drawings, and original design drawings.

It is noted that this DBDPA is considered an approximation of runoff and flows since storms rarely follow ideal patterns and other factors such as ground cover and infiltration may vary with time or from assumed conditions. Actual flows may be higher or lower than calculated. The intent of any hydrologic/hydraulic analysis is to provide a reasonably dependable and consistent approximation of runoff and routing flow characteristics. It should also be remembered that floods larger than the 100 year design flood can and will occur.

The flow depths given in this report are also considered an approximation because they are based on assumed, idealized, typical channel sections and do not account for any possible backwater.

Users of previous City of Rapid City DBDP's prepared with CUHP and UDSWM will notice this report does not contain pages and pages of computer output like the previous plans contained. This is because HMS has only limited reporting capabilities and those are generally for summary tables. As such it was necessary to prepare various spreadsheets to better summarize the input and output data. Copies of the actual computer models with all data are also provided with this deliverable package

# 2. SUPPORTING LITERATURE AND DATA

## 2.1 INTRODUCTION

This section provides a general listing of the literature and other data that was using to prepare this DBDPA. The below listing should not be considered as the full, all inclusive, list of information. Additional supporting literature and data not listed below would include such items as user manuals for various software, field reconnaissance, engineering judgement, etc.

## 2.2 PREVIOUS DBDP STUDIES

### ORIGINAL DBDP

The original DBDP for this study area is entitled "County Heights Drainage Basin Design Plan" and was prepared by Davis-Atkins & Associates in December 1990. This report is on file at the City of Rapid City. This document has served as the general guidance document for storm water improvements and developments that have occurred in the basin.

The 1990 DBDP and additional items described below were prepared for Pennington County by the now defunct Western Pennington County Storm Drainage and Flood Management Commission.

### PRELIMINARY DESIGN REPORTS

Ferber Engineering prepared the "Report on Racetrack Draw and County Heights Drainage Basins Preliminary Designs" in June 1996. That report presents preliminary design, including options, of various major conveyance elements in the study area. The report also included discussion related to updated design flows as a result of preliminary designs and provided alternate routes for the County Heights major drainage between Albert Lane and Rapid Creek. The flows from that report that were used for project design are different than those tabulated in the original DBDP. No official amendments to the original DBDP were found related to this June 1996 report.

Ferber Engineering prepared a report entitled "Racetrack Draw and County Heights Drainage Basin Projects – Transition Phase From Preliminary Design to Final Design" in July 1996. That report includes a hydraulic schematic showing a general layout of drainage facilities and provided Pennington County with cost estimates. It is worth noting that the report included a schematic that changed the routing south of Albert Lane to Rapid Creek to follow the major drainage paths that were subsequently constructed. This routing was a significant change from that described in the original DBDP.

It is assumed the July 1996 Report was considered an Amendment to the DBDP because it includes a Staff report to the County Commission recommending approval of the preliminary drawings and report.

An annotated copy of the July 1996 routing schematic along with UDSWM printouts was found in the Appendix of the 1990 DBDP on file at the City of Rapid City. The Hydraulic Schematic has an annotation date of February 12, 1999. The UDSWM printouts are dated April 1996. Printouts were not included for CUHP changes that were necessary for the new routing. There was also a handwritten note with the schematic listing the name of computer files and stating "here is the update to the County Heights Drainage Plan to reflect the as built conditions".

As noted above the 1996 routing significantly changed the major drainage path from Albert Lane to Rapid Creek. The 1996 Reports did not address any changes to the 1990 DBDP in the "Longview

Road/Trailwood Village" sub-basins continuing downstream to Rapid Creek as a result of the changed major flow path. The only data in those reports is the schematic which illustrates the assumed flow direction from those basins continuing south across Highway 44 and then through property where no channel currently exists same as the original 1990 DBDP.

No sub-basin boundary maps were found for any of the routing changes described above.

## 2.3 STUDIES BY DEVELOPMENT ENGINEERS

To date the detention ponds located north of Twilight Drive have been constructed by developers. Listed below are the known design reports for these ponds.

*"County Heights Drainage Basin Detention Cell 103 Design Summary"* dated January 26, 2001 and prepared by Dream Design International. This is the original design report for Cell 103.

*"Report of Hydrologic and Hydraulic Calculations Detention Cell #101 of County Heights DBDP"* dated January 21, 2008 and prepared by Sperlich Consulting. This report included calculations for the design of Detention Cell 101. It also include calculations for Detention Pond #104 which was a new pond proposed by this same report.

*"County Heights Drainage Basin Modeling Analysis"* dated February 26, 2008 and prepared by Dream Design International. This updates the January 26, 2001 report to correct errors that were made to earlier CUHP model input.

*"Report of Hydrologic and Hydraulic Calculations HEC-HMS Model for Homestead Subdivision, Phase 1"* with final revision date December 15, 2009 and prepared by Sperlich Consulting. This report addresses the final design for the Detention Cells 101 and 104. This report is based on HMS rather than the CUHP/UDSWM models that were in the all previous work in the basin.

## 2.4 CONSTRUCTION / AS BUILT PLANS

The following construction and/or as-built plans were used to obtain data for ponds, channels, and culverts.

*"Reservoir Road and Longview Road - Grading, AC Surfacing, Curb and Gutter, Storm Sewer, & Pavement Markings*," South Dakota Department of Transportation Projects P-PH 1610(17) and P1612(1), prepared by CETEC, 2009.

*"SD Highway 44 - Grading, Structures, PCC Pavement, Curb and Gutter, Storm Sewer, Roadway Lighting, Pavement Markings, & Permanent Signing*," South Dakota Department of Transportation, Project P 0044(52)50, 2007.

*"SD Highway 44 – Grading and PCC Pavement for Center Turn Lane, PCC Pavement Repair, and ROW Plans*," South Dakota Department of Transportation, Projects P 0044(24)49 and 0444-452, 2002.

*"Twilight Drive - Grading, Surfacing, Curb and Gutter, and Storm Sewer*," South Dakota Department of Transportation Projects P-PH 1555(01), prepared by Clark Engineering, 1996.

*"Rapid Valley Drainage Improvements, Racetrack Draw and County Heights Basin Outlet Channels,"* Ferber Engineering Company, October 1996

_“County Heights South Final Design,”_ Project J98-111, Ferber Engineering Company, January 2000

_“Rapid Valley Concrete Box Culverts,”_ Pennington County Highway Department, December 1996

_“Covington Street Grading, Surfacing, Storm Sewer, and Curb and Gutter,”_ Project 420407-03, Pennington County Highway Department, February 2003

_“County Heights Detention Cell #100 DDI Job#00-0087,”_ Dev. 427, Dream Design International, November 2000

_“County Heights Drainage Basin Detention Cell #103,”_ Dev. 436, Dream Design International, February 2001

_“East Middle School,”_ Upper Deck Architects Inc., November 2010

_“Windmere Subdivision Phase 3,”_ Dev. 397, Centerline, April 2000

_“Big Sky Subdivision Homestead Street,”_ Dev. 621, Dream Design International, September 2003

_“Big Sky Subdivision Phase III,”_ Dev. 356, Centerline, April 2000

_“Big Sky Subdivision Phase IV,”_ Dev. 389, Dream Design International, April 2000

_“Big Sky Subdivision Phase V,”_ Dev. 469, Dream Design International, August 2001

_“Big Sky Subdivision Phase VII,”_ Dev. 482, Dream Design International, December 2002

_“Valley Ridge Subdivision Phase III,”_ Dev. 423, Dream Design International, October 2000

_“Valley Ridge Subdivision Phase VI,”_ Dream Design International, May 2001

_“Trailwood Village Phase 5 Grading Plan,”_ Dev 311, Renner & Sperlich Engineering, April 1998

_“Trailwood Village Phase 6 Grading Plan,”_ Dev. 385, Renner & Sperlich Engineering, March 2000

_“Trailwood Village Phase 9,”_ Dev 680, Renner & Sperlich Engineering, April 2004

_Plans for Dalcom Oil Company,”_ Sperlich Consulting, Inc., March 2005

_Plans for Homestead Subdivision Phase 1,”_ Dev.901, Sperlich Consulting, Inc., January 2008

## 2.5. CITY DIGITAL INFORMATION AND MISCELLANEOUS DATA

The following City of Rapid City digital data was obtained for the study.

*City of Rapid City 2008 Aerial Orthophotography - GIS*

*City of Rapid City 2 Foot Aerial Contours –GIS (1929 Datum)*

*City of Rapid City GIS Storm Sewer Layer*

The following miscellaneous City of Rapid information was obtained for the study.

*Storm Sewer Map – Circa 2001*

*City of Rapid City Airport Neighborhood Future Land Use Map – February 2005*

*City of Rapid City Elk Vale Neighborhood Future Land Use Map – October 2010*

*City of Rapid City SE Connector Future Land Use Plan – May 2011*

## 2.6. FEDERAL DATA SOURCES

The following Federal sources were used to obtain information for the study. Information from these sources was obtained via the internet.

*NRCS WEB Soil Survey*

*FEMA Floodplain Map Panel 792 dated June 2013*

*USFWS National Wetland Inventory*

## 2.7. TECHNICAL REFERENCES

Following is a list of technical references that were reviewed for the study.

*"Rapid City Infrastructure Design Criteria Manual"*, City of Rapid City, 2012

*"Rapid City Drainage Criteria Manual,"* City of Rapid City, 1989

*Rapid City Stormwater Quality Manual,"* City of Rapid City, 2009

*"Urban Storm Drainage Criteria Manuals,"* Urban Drainage and Flood Control District, Denver, Colorado, 2011

*"Open Channel Hydraulics,"* Ven Te Chow, 1959

*"Handbook of Hydrology,"* David Maidment, 1992

*"Introduction To Hydrology 5th Edition,"* Warren Viessman & Gary Lewis, 2002

*"Hydrology Handbook 2nd Edition,"* ASCE Manuals and Reports on Engineering Practice #28, American Society of Civil Engineers, 1996

*"Hydrologic Modeling System HEC-HMS User's Manual Version 3.5 - CPD-74A"* United States Army Corps of Engineers Hydrologic Engineering Center, August 2010

*"Hydrologic Modeling System HEC-HMS Release Notes Version 3.5,"* United States Army Corps of Engineers Hydrologic Engineering Center, August 2010

*"Hydrologic Modeling System HEC-HMS Technical Reference Manual – CPD-74B,"* United States Army Corps of Engineers Hydrologic Engineering Center, August 2010

*"Hydraulic Design of Highway Culverts – HDS5,"* Federal Highway Administration, May 2005

*"Flood-Runoff Analysis, EM1110-2-1417,"* United States Army Corps of Engineers, August 1994

*"Corps of Engineers Wetlands Delineation Manual, Wetlands Research Program Technical Report Y-87-1,"* United States Army Corps of Engineers Environmental Laboratory, January 1987

*"Interim Regional Supplement to the Corps of Engineers Wetland Delineation Manual: Great Plains Region,"* United States Army Corps of Engineers – Engineer Research and Development Center, March 2008

*"Storm Water Management Model Users Manual Verison 5.0 – Table A.2,"* U.S. Environmental Protection Agency, November 2004

*"Methodology for Estimating the Effective Impervious Area of Urban Watersheds,"* Technical Note 58, Roger C. Sutherland, P.E., Watershed Protection Techniques, Vol. 2, No. 1, Fall 1995

*"National Engineering Handbook – Part 630 Hydrology,"* United States Department of Agriculture, National Resources Conservation Service

*"Soil Survey of Custer and Pennington Counties, Prairie Parts, South Dakota,"* United States Department of Agriculture, National Resources Conservation Service

# 3. GENERAL BASIN INFORMATION

## 3.1 STUDY AREA

As noted earlier the County Heights Drainage Basin incorporates 1,702 acres. The County Heights Drainage Basin is shown on Figure 1. Figure 2 shows the sub-basins used for modeling of the overall basin.

Of special note is that in certain instances the study area does not coincide with the overall basin boundaries that appear apparent from the contours. These areas are denoted on Figure 3 and are further discussed below.

The 13 acre basin shown as North Basin Transfer Sub Area appears from contours to drain south across Twilight Drive and into the County Heights Study Area. However, there is a large storm sewer in Twilight Drive that intercepts flow from this sub-area and conveys it west to the Racetrack Basin. It is possible some flow may not be intercepted and spill over into the County Heights Study Area; however, that potential spill overflow has been judged insignificant to the study. The original 1990 County Heights DBDP assumed this area would drain into the County Heights study area. The storm sewer was constructed during the Twilight Drive Reconstruction circa 1997. It is assumed this transfer, including impacts to the Racetrack Basin, was approved by the appropriate authorities and therefore no analysis or recommendation has been made to redirect the flow back to the County Heights basin.

In regards to the North Basin Transfer Sub Area it is also noted that under existing conditions additional flows will reach the storm sewer. These would be Plateau Lane flows that bypass the main drainage channel at the north edge of this area. This additional bypass flow is intended to be captured as described in the Element 11 recommendations in Section 7 of this report.

The area shown as the West Basin Transfer Sub Area, approximately 43 acres, appears from contours to be in the County Heights Study Area either by direction of flow or due to interception by the Hawthorne Ditch. However, a basin larger than this was described in the 1990 County Heights DBDP and 1990 Racetrack DBDP as being an area that would have flows captured and directed to the Racetrack Basin.

The redirection to the Racetrack Basin was noted as being necessary due to capacity issues related to the Hawthorne Ditch. The area described in 1990 was defined as Sub-basin 110 in the Racetrack DBDP. This redirection would have been a 100 year flow diversion.

The 1990 County Heights DBDP has narrative that recommended diversion in addition to Racetrack Sub-basin 110. This additional diversion area was the west half of Sub-basin 3 in that study. Sub-basin 3 in that study more or less correlates with sub-basin 8W in this DBDPA study. This area would have been redirected to the Racetrack Basin via a proposed storm sewer from Covington Street. The storm sewer was to be sized to redirect the 10 year flow. A storm sewer has now been constructed by Pennington County along Covington Street; however, the storm sewer has directed flows east into the Hawthorne Ditch rather than to the Racetrack Basin as previously planned.

The West Basin Transfer area in this Amendment is now smaller than previously proposed due to the review of current contours and because of the way Pennington County installed the Covington Street storm sewer.

Under current conditions the West Basin Transfer Sub-Area will actually drain into the County Heights Basin, mostly by interception by the Hawthorne Ditch. However; for the purposes of this Amendment and consistent with previous studies and direction of City Staff, it was assumed this flow does not enter the study area. It was beyond the scope of this study to make recommendations for methods of flow capture and conveyance from the County Heights West Basin Transfer Sub-Area to the Racetrack Basin. Rather, this Amendment is simply based on the assumption that the flow will be redirected at some date.

It is also noted that the east boundary of the original DBDP was more or less along Reservoir Road. There are actually areas east of Reservoir Road that drain into this study area.

## 3.2 GENERAL DESCRIPTION OF BASIN DEVELOPMENT

Current development in the study area is predominantly residential. The residential development is mostly single family although there are also areas of apartments, mobile home parks, and townhouses. Other uses such as schools, churches, industrial, and commercial are also found in the study area.

The City of Rapid City has developed future land use plans that include the study area. These land use plans are the Elk Vale Neighborhood Future Land Use Plan, the SE Connector Future Land Use Plan, and the Airport Neighborhood Future Land Use Plan. Maps of these land use plans are included as Figures 4, 5, and 6 respectively.

This study assumes future land use in the basin will be as indicated on the future land use maps. Further discussion related to future land use is found in Section 5.

## 3.3 FEMA FLOODPLAIN

A FEMA floodplain is located along Rapid Creek at the southern end of the study area. The FEMA floodplain boundaries are shown on Figure 1. Floodplain information was obtained from FEMA floodplain maps dated June 2013.

There are no mapped floodplains along any of the tributary channels in the study area. The only mapped floodplain is along Rapid Creek.

It was beyond the scope of the project to perform any floodplain analysis, either in the FEMA mapped area or along any of the tributary channels.

## 3.4 IRRIGATION DITCHES

Three main irrigation ditches are located in the study area. These ditches are the Hawthorne Ditch, Murphy Ditch, and the Little Giant Ditch. The ditches are indicated on Figure 1. A more extensive discussion of the irrigation ditches is found is Section 11.

## 3.5 WETLANDS

Wetlands are found within the study area. One specific concern related to wetlands is that several of the major constructed channels were designed under the assumption of a maintained grass channel. These channels have now become overgrown with wetland type vegetation such as cattails, marsh grasses, trees, and the like. The wetland vegetation in these channels significantly reduces the channel capacity.

A more extensive discussion of wetlands is found is Section 4. The individual element design discussion and storm water quality sections also include discussion related to wetlands.

## 3.6 WATER PRESSURE ISSUE

The Elk Vale Future Land Use plan describes a water pressure issue in the northern portion of the study area. That plan notes that any development above the 3300 elevation contour will "require either a lift station or water tower if the developer intends to provide municipals services to the area." This general location of the 3300 elevation contour location is indicated on Figure 1.

This water pressure issue is included in the report for information only. It is assumed this issue will delay development in the upper reaches of the basin. It is beyond the scope of the project to perform any water system studies or provide recommendations.

## 3.7 MAJOR STREETS

Several major streets are located in the study area. The major streets are shown on the previously noted future land use plans. Major streets are also shown on the City of Rapid City Major Street Plan which is included as Figure 7.

Depending on location, a major street may currently be under the jurisdiction of the City of Rapid City, Pennington County, or the State of South Dakota.

## 3.8 MINOR DETENTION PONDS

There are a number of minor or small onsite detention ponds located in the study area. One such pond that is known to exist is the onsite pond at the school site north of Homestead Street. Another such pond is a small mostly ineffective pond near Reed Court. Other ponds may exist.

A previous onsite pond was located at the southeast corner of Shaw Court and Reservoir Road. This pond was abandoned during the reconstruction of Reservoir Road.

Ponding areas created by roadway embankments have been ignored in this study except for the regional Detention Pond 104 which is created by Homestead Drive. Other small ponding area created by roadway embankments will create only incidental levels of detention that are beyond the accuracy of the study.

**Additional small onsite ponds are expected to be constructed in the future. These ponds are needed to meet the RCIDCM requirement of maintaining runoff to historic conditions prior to implementation of all improvements in this DBDPA. These ponds are also expected to be used to meet Water Quality Capture Volume requirements for stormwater quality treatment.**

All existing and future minor, small, and onsite ponds have been ignored in this study because (1) the ponds are too small to be accurately included in the overall analysis, (2) history shows some small ponds may become abandoned, (3) private ponds will likely not be maintained to design conditions, and (4) uncertainty as to where future small ponds would be located.

Unless otherwise noted in detailed discussions the only detention ponds included in the HMS analysis are the regional ponds described in Section 8 of this report.

## 3.9 PROBLEMS AND NEEDS

As noted earlier the study is limited to major drainage only. It was beyond the scope of the project to hold open houses or to make mailings/contacts with the landowners in the area. Therefore, the identification of problems and needs in the basin was limited to (1) visual observations, (2) comments from City staff, and (3) analysis of the study results. Following is a brief outline of the major problems identified in this study.

- Lack of documentation between new studies/designs and the original DBDP. This was discussed in detail in Section 2.

- Irrigation companies, especially the Hawthorne Ditch Company, have expressed concerns about the ditches being used for stormwater conveyance and the resulting problems with overflows, clogging of roadway crossings, etc.

- Lack of access routes to channels and detention ponds for maintenance

- Several channels are taking on the characteristics of wetlands which reduce channel capacity below the original design.

- Most of Sub-basin 8E drains west to major drainage Element 14. The original DBDP and the original design of these Elements assumed this area would drain to the east rather than to Element 14. Another portion of Sub-basin 8E drains east to Element 14 whereas the original DBDP assumed this area would drain west to Plateau Lane. This issue, combined with wetland channel bottoms and the changed location of Detention Pond 101 (as discussed below) has led to increased flows and thus capacity problems beginning at Element 14. Sub-basin 8E is 26 acres in size. Figure 8 illustrates the location of this flow direction issue.

- Detention Pond 101 was constructed just downstream of Homestead Street. The original DBDP recommended this pond be constructed farther down the basin at a location more or less where Avenue A crosses the channel. The detention pond was relocated at the direction of the now defunct Western Pennington County Storm Drainage and Flood Management Commission because of the availability of donated land for the pond; however, no studies were prepared to determine the consequences of the pond relocation. The result of this relocation is that about 40 acres of drainage area is not routed through a pond as originally planned. This pond relocation, combined with the flow direction of Sub-basin 8E (as discussed above) and wetland channel bottoms has led to increased flows and thus capacity problems beginning at Element 14. Figure 8 illustrates the various locations of Detention Pond 101.

- Several improvements are recommended to the major conveyance elements and various ponds. These are tabulated and described in the respective report sections for those items.

# 4. WETLANDS

## 4.1 BACKGROUND

Section 404 of the United States Clean Water Act Amendment authorizes the USCOE to issue permits for the discharge of dredged or fill material into the "Waters of the United States". Wetlands fall under the definition of "Waters of the United States." However, in addition to wetlands, there are many other "Waters of the United States" where the USCOE asserts jurisdiction under Section 404.

Section 404 describes wetlands as areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. In simple terms this means for an area to be a USCOE wetland it must have hydric soils, and have wetland hydrology, and have hydrophytic vegetation.

It is beyond the scope of this report to provide a detailed description of the definitions of "Waters of the United States" and regulatory requirements of Section 404 of the Clean Water Act or to provide a description of all jurisdiction that can be asserted by the USCOE. Only the USCOE can make the final regulatory confirmation as to whether areas are "Waters of the United States" wetlands or if they have any other jurisdiction related to Section 404 of the Clean Water Act.

## 4.2 POTENTIAL WETLANDS IN STUDY AREA

The intent of the potential wetland identification in this study is simply to illustrate the potential that wetlands or other USCOE jurisdictional areas may be present in the study area.

Figure 9 illustrates the areas judged as having the potential to be wetlands or as otherwise being under USCOE jurisdiction. Notes are included on the figure as to why the specific item is included. It needs to be understood that some of the areas shown as being potential wetlands/jurisdictional, or identified as wetlands on the NWI, actually may not be wetlands or under any USCOE jurisdiction. It also needs to be understood that there may be wetlands or jurisdictional areas in addition to the potential areas shown.

Identification of potential wetlands along Rapid Creek was beyond the scope of work.

Areas that have been judged as having the potential of being wetlands or under USCOE jurisdiction were identified using offsite desktop methodology to look for indicators of hydrophytic vegetation, hydric soils, and wetland hydrology. Detailed field investigation to identify these indicators was beyond the scope of work. Contact with the USCOE for confirmation of wetlands or other jurisdictional areas was beyond the scope of work.

The offsite methods included:

- Identify wetland areas from U.S. Fish and Wildlife NWI maps. These were obtained at the NWI website.
- Transpose ponds, irrigation canals, and "blue" intermittent stream lines from the USGS Rapid City East Quadrangle Map (1978).

- Review 2008 and 2010 NRCS aerial photos for signs of wetlands such as hydophytic vegetation, surface water, saturated soils, flooded crops, stressed crops due to wetness, green vegetation, etc.

- Identification of primary drainage paths. Primary drainage paths were included in the review because the USCOE may assert jurisdiction over drainage tributaries by applying a significant nexus standard even if the tributary appears "dry".

- Identify areas of hydric soils and area of soils with hydric inclusions using NRCS soil maps and soil lists.

Desktop methods were supplemented with cursory site observations to identify areas with easily observable hydrophytic vegetation. For the purposes of this study, easily observable hydrophytic vegetation was defined as plants, such as cattails, reeds, and cottonwoods, that a layperson could be expected to associate with a wetland or water affected area. A detailed study to determine the types and prevalence of actual hydrophytic vegetation was not made.

Only a limited review for wetland hydrology indicators was performed. The only hydrology indicators used for this study were observations for standing or flowing water. A trickle to base flow was observed in those channels noted as such. It appears a significant amount of the trickle and base flow may be from irrigation ditch leakage or irrigation ditch overflow into the diversion structures. It is also believed there is a groundwater or sump pump contribution to the base flows because a small base flow was observed at the Highway 44 box culvert and the Longview Road box culvert during the fall and in the winter during non-melting periods.

Project specific investigations, identifications, and USCOE jurisdictional determinations for Section 404 will be necessary at the time specific projects are brought forth. **The information contained in this report section and on Figure 9 is not considered as an official identification of wetland/nonwetland areas, "Waters of the United States," or any other USCOE jurisdictional/nonjurisdictional area.**

# 5. SUB-BASIN HYDROLOGY

## 5.1 METHODOLOGY

Sub-basin hydrology input data was developed following recommendations and requirements of the RCIDCM and using engineering judgment and reasoning. Complete tables of input data are included in Appendix A and Appendix B.

Appendix A contains the sub-basin input data for Existing Land Use Conditions. Appendix B contains the sub-basin input data for Future Land Use Conditions.

## 5.2 SUB-BASIN DELINEATION

Sub-basins were established using engineering judgment in order to provide a reasonable subdivision to reflect slopes, cover, and land uses. Sub-basin boundaries were determined using aerial contours and field observations.

DBDPA sub-basin boundaries do not match the boundaries in the original DBDP for various reasons including: (1) up to date and better mapping information, (2) as built locations of detention ponds, (3) changes in overall study basin boundary, and (4) different and/or additional design points desired.

Sub-basin boundaries and identification numbers are shown on Figure 2.

## 5.3 PRECIPITATION

The following meteorology methods and data were used as required by the RCIDCM.

- Use synthetic frequency storm option with 2 hour rainfall duration.
- Use five minute time step for development of design storm.
- Storm peak shall occur at the first quartile.

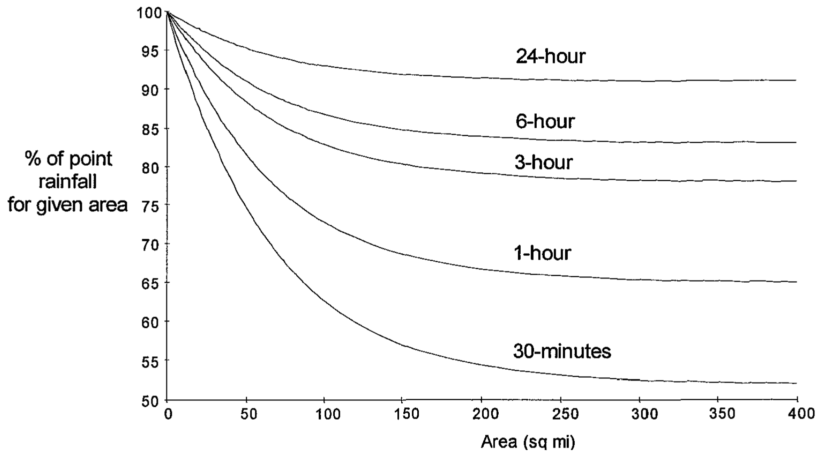
Table 1 below provides rainfall data that was used in the study. This data is from the RCIDCM.

**TABLE 1**

FREQUENCY STORM RAINFALL AMOUNTS (INCHES)

| DURATION | 2 YR | 10 YR | 100 YR |
|---|---|---|---|
| 5 Min | 0.36 | 0.53 | 0.79 |
| 15 Min | 0.69 | 1.04 | 1.57 |
| 60 Min | 1.05 | 1.86 | 2.95 |
| 120 Min | 1.20 | 1.98 | 3.06 |

HMS applies an area correction factor to reduce the specified precipitation depths. The reduction is based on the chart on the following page which is from the HMS User Manual. An HMS input option triggers if the reduction is by individual sub-basin or for the overall basin.




HMS Version 3.5, which was used for this study, does not have an input option to eliminate the precipitation reduction. However, for the purposes of this study an alternate input area of 0.01 square miles was used as a workaround to eliminate the precipitation reduction rather than using the actual study area of 2.66 square miles.

The precipitation reduction would have been minor due to the small 2.66 square mile overall basin size; nevertheless, it was judged reasonable to use the alternate area because (1) the RCIDCM has no discussion regarding aerial reduction of rainfall and (2) the HMS Technical Manual states point precipitation values should be used without reduction for areas up to 9.6 square miles, and (3) a small safety factor is provided by using the point precipitation value without reduction. The value of 0.01 square miles maintains the same hyetograph for all basins and results in a depth area reduction so small as to not be noticeable. The resulting precipitation in the HMS calculations is thus the 120 minute values given in Table 1 without any reduction.

## 5.4 PRECIPITATION LOSSES

### 5.4.1 INITIAL LOSS

Initial loss in HMS is the precipitation that is lost on pervious surfaces by such means as being captured by leaves and vegetation, ground cover such as thatch or duff, and minor surface depressions.

Initial Loss was part of HMS Green-Ampt input data in earlier versions of HMS. This has been removed from Green-Ampt input in HMS Version 3.5. The Release Notes for HMS Version 3.5 describe the Initial Loss as being input as Simple Canopy Storage. Models that were created with earlier versions of HMS will have the Initial Loss parameter automatically moved to Simple Canopy Storage.

The RCIDCM recommendations for Initial Losses on Pervious surfaces were judged reasonable for the project area and were thus used in this study. Initial abstraction was thus assumed as 0.40" for Open Fields and 0.35" for Lawn and Grass areas. Many mature trees are apparent in the older developed areas of the study area; however, any additional initial loss that may occur from tree canopy interception has been ignored.

Initial storage content of the Canopy Storage was assumed as zero under the assumption that all Canopy Storage was evacuated by evapotranspiration prior to the design storm.

HMS assumes 100% runoff from impervious surfaces and therefore no initial retention loss will occur on the impervious surfaces. There is no method in HMS to input initial loss on impervious surfaces even though such losses are known to occur. It is commonly assumed that initial loss on impervious surfaces ranges from 0.05 inches to 0.30 inches. The only method in HMS to account for losses on impervious surfaces is for them to be accounted for in the Mapped Impervious Area (MIA) to Effective Impervious Area (EIA) reduction. This is further discussed in Section 5.5.

Prorating to create "dummy" initial loss values so that impervious losses are added onto the pervious loss value was considered but ultimately rejected because (1) this was judged to be unreasonably complex, (2) is not documented in any manual, (3) would result in false input data being applied towards infiltration calculations, and (4) would duplicate losses created by reducing MIA to EIA.

### 5.4.2 INFILTRATION LOSS

The RCIDCM requires the use of the Green-Ampt method to determine infiltration losses. Input data for the Green-Ampt infiltration method are described in the RCIDCM as being Initial Loss, Moisture Deficit, Suction Head, Conductivity, and Imperviousness.

HMS Version 3.5 has changed the Green-Ampt input parameters from those described in the RCIDCM as follows. Initial Loss is no longer input as Green-Ampt data as described in Section 5.4.1. The Moisture Deficit parameter has also been replaced with two parameters: saturated content and initial content.

Green-Ampt input data is a function of soil types and requirements of the RCIDCM. The soil types were determined from NRCS soil maps. Figure 10 is a map showing the NRCS soil types.

Saturated Content of soil has been assumed to be equal to the effective porosity of the soil. The HMS User Manual describes Saturated Content as often being assumed to be the total porosity of the soil. However, because some soil pores are not available for infiltration and to be consistent with the RCIDCM, effective porosity has been used for the Saturated Content. Values for Effective Porosity for the various soil types were taken from Table 4-4 in the RCIDCM.

The initial water content gives the initial saturation of the soil at the beginning of calculations. Per discussion in the RCIDCM the initial water content has been assumed to be 50 percent of the field capacity for any given soil type. Field capacity values are not available in the RCIDCM and were thus determined from a table of values in the EPA SWMM User's Manual.

Values for Suction Head for the various soil types were obtained from Table 4-4 in the RCIDCM.

Hydraulic conductivity values from Table 4-4 in the RCIDCM were used with the following adjustments. The Green-Ampt hydraulic conductivity values in the RCIDCM are for "bare" soil conditions although not described as such on the table or in the text. A review of the Maidment Handbook of Hydrology, ASCE Hydrology Handbook and other references leads to the conclusion that the RCIDCM values are for "bare" soil. As described in the same references, hydraulic conductivity for vegetated areas can be estimated as being twice the value of "bare" soil data. The contributing drainage basins appear fully vegetated, except for areas determined as being impervious; therefore, the hydraulic conductivity values from Table 4-4 were doubled for use in this study. There may be small isolated areas of less than 100% vegetation but these were judged insignificant in size, otherwise flow over well drained soils, or are well drained themselves.

It is recognized that the RCIDCM Green Ampt soil data was obtained from studies of non-urban areas. It is unknown if urbanization in the study area has changed any of the Green Ampt data from those outlined in the RCIDCM. This was discussed during project scoping and direction was given that the project was not to include any attempt to quantify changes to soil characteristics as a result of urbanization. Nevertheless, as good Engineering Practice, the reasonableness of the RCIDCM data was given thought and the following points would seem to support the assumption, and scoping instructions, that urbanization would not make significant changes to the Green Ampt soil data:

- Older developments in the basin appear to have been constructed with very little grading performed.
- Full depth topsoil stripping and replacement has likely been used during all developments.
- Plant rooting and opening of pores from long term vegetation establishment.
- Many soil types have thick upper horizons in which case the NRCS soil types remain the same after grading.
- Undeveloped upper reaches of the study area are expected to have only minor grading due to the anticipated lot size.
- About 14% of the study area is a Gravel Loam which was assumed to be a Loam for Green Ampt data. By its nature Gravel Loam would actually have infiltration values higher than Loam.
- As post construction storm water requirements continue to become implemented, credits for reduction in land disturbance or appropriate topsoil thickness/amendments to promote infiltrate can reasonably be expected to occur. In fact requiring appropriate topsoil and/or amendments in developments is recommended as a requirement for Post Construction Storm Water Quality Management.
- Future EPA requirements may require projects retain runoff volume for water quality level storms to predevelopment volume. This would likely then lead to the use of LID techniques including appropriate topsoil and amendments to promote infiltration. Use of appropriate depths of topsoil and amendments is described as a recommendation in the Water Quality Section of this report.
- It is noted that several areas in the study basin were already urbanized when the NRCS maps were prepared.

## 5.5 SUB-BASIN IMPERVIOUSNESS

### 5.5.1 MAPPED IMPERVIOUSNESS AREA DISCUSSION

The following criteria and assumptions were made for determination of Mapped Impervious Area (MIA). The overall sub-basin MIA values were then reduced to Effective Impervious Area (EIA) for input into HMS.

Future Land Use was assumed to be per City of Rapid City future land use plans. The plans were previously identified in Section 3.2 and are included as Figures 3, 4, and 5. These plans are further described as:

- Elk Vale Neighborhood. - The future land use map for this neighborhood is dated October 4, 2010. The vast majority of the study area is covered by this map. It is noted that the future Park Site north of Homestead Street and west of Reservoir Road was assumed as residential per the alternative uses identified on the map.

- SE Connector Neighborhood. - The future land use map for this neighborhood is dated May 16, 2011. This map was used for the portions of the study area located south of Highway 44.

- Airport Neighborhood. - The future land use map for this neighborhood is not dated. This map was used for the portions of the study area that are located east of the Elk Vale Neighborhood map.

Land use for the existing conditions analysis is based on development that existed in the basin at the time of the study. The existing land use was estimated from GIS aerial photography and related to other assumptions described in this section.

Described below are further assumptions that were made for estimating MIA for various land use types.

#### 5.5.1.1 Low Density Residential

The RCIDCM recommends 45% impervious be used for Single Family Residential and for ½ Acre or Larger Lots. This value was judged high in the study area and an analysis of the MIA for existing single family areas was made for verification purposes. Twelve (12) sample locations ranging in size from 3.2 acres to 8.6 acres were selected and impervious areas measured from aerial photography. Density of the sample sites ranged from 1.7 to 3.8 houses/acres. In older subdivisions, gravel driveways were assumed as being impervious for this MIA study.

A best fit line of the resulting MIA values was then prepared. The resulting minimum MIA of the best fit line is 28% for 1.7 houses/acre. At 2 houses/acre the resulting MIA is about 30% or well below the 45% recommended in the RCIDCM for ½ acre lots.

The resulting maximum MIA of the best fit line is about 37% for a density of 3.8 houses/acre. This value is also well below the 45% recommended in the RCIDCM for single family areas.

The values were also plotted against the recommended impervious values for single family housing density chart in the 1989 Rapid City Drainage Criteria Manual. All data, except one point, and the best fit line are below the curve in that 1989 chart.

The best fit line determined by this process was then used for estimating imperviousness of single residential areas for this study. This graphed line, along with the curve from the 1989 RCDCM is included as Figure 11.

Imperviousness for existing LDR areas is based on the developed best fit curve using the actual housing density. Imperviousness for future LDR imperviousness assumes a density of 3.4 houses/acre which more or less the average of the existing LDR development. Use of this imperviousness data and assumptions was approved by City staff during an early review meeting.

In the case of the PRD area with a density of 1.5 units/acre, the best fit line was extrapolated to 27% imperviousness. It is noted this is the only case where the best fit line results in imperviousness that is greater than the 1989 curve value.

No adjustments were made in the LDR land use areas to account for isolated uses such as churches, utility stations, home occupations, etc.

### 5.5.1.2 Land Use Other Than Low Density Residential

Unless otherwise described below the recommended values of imperviousness in the RCIDCM were used for land uses other than low density residential. It is noted the RCIDCM impervious values for the highly impervious land use types may be high especially as Post Construction Stormwater Quality and LID requirements become more prevalent and more restrictive EPA Storm Water regulations are imposed; nevertheless, the RCIDCM recommended values were used.

General Commercial impervious was assumed at 95% per the RCIDCM recommendation for Business/Commercial.

Office Commercial with PCD was assumed at 95% imperviousness per the RCIDCM recommendation for Business/Commercial.

Light Industrial impervious was assumed at 80% per the RCIDCM.

Neighborhood Commercial, either existing or future, assumes 70% imperviousness in accordance with the Business/Neighborhood value in the RCIDCM.

Medium Density Residential impervious was assumed at 60%. This is the average impervious of detached (50%) and attached (70%) multi unit land use in the RCIDCM.

Planned Unit Development, PUD, imperviousness is based on combination of existing and proposed use within each PUD.

Highway 44 Right of Way was made a separate land use type. Normally, street and highway right of ways are included in the adjacent land use. Highway 44 has an overwidth right of way, including a railroad right of way, thus the need for a separate land use. Imperviousness of the Highway 44 right of way is based on the width of existing pavement within the overall right of way.

Schools assume 50% imperviousness in accordance with the School value in the RCIDCM. It is noted that MIA of the two school sites, including the current expansion of the East Middle School, was determined using aerial photos and design drawings. The MIA was actually closer to 35%. However, it was judged appropriate to use the 50% value to account for expansion at the sites similar to what has occurred at other schools for items such as Community Gyms, added parking, more classrooms, etc.

Impervious of the two existing Mobile Home Parks is based on actual MIA as estimated from the aerial photography for these two locations. MIA analysis was necessary because the RCIDCM does not have a recommended value for Mobile Home Parks.

Public Land use, except for schools, was divided into two categories. The area of Public Land use north of Highway 44 is for the fire station at the Reservoir Road and Highway 44 for which impervious was assumed at 80%.

The second Public Land use is that area south of Highway 44. Imperviousness of this area was assumed at 7% per the RCIDCM recommendation for Parks/Cemeteries. This impervious also coincides closely with imperviousness for about 0.7 unit/acre low housing density on the 1989 RCDCM housing impervious curve.

5.5.2 EFFECTIVE IMPERVIOUS AREA FOR HMS INPUT

Hydraulically connected impervious area as defined by the RCIDCM is the impervious area that is hydraulically controlled by direct runoff to a curb and gutter and subsequent channel drainage. Runoff from impervious area that is directed back over pervious area must be allowed enough time to infiltrate to not be considered hydraulically connected. The City of Rapid City has an objective to promote reduction in hydraulically connected impervious area. Reducing hydraulically connected impervious area reduces the runoff peak, runoff volume, and pollutant load of runoff.

Effective impervious area (EIA) is the portion of the mapped impervious area (MIA) within a basin that is directly connected to the drainage system. EIA includes street surfaces, paved driveways connecting to the street, sidewalks adjacent to curbed streets, rooftops connected to impervious areas, parking lots, etc. EIA is usually measured as a percentage of total basin or sub basin area. As commonly understood in the industry and as stated in the RCIDCM "*In traditional urban runoff modeling, the EIA for a given basin is usually less than the MIA. However, in highly urbanized basins, EIA can approach and equal MIA values.*"

The RCIDCM discusses the Sutherland Equation as being used for DBDP's as a method to estimate a reduction in MIA to EIA. The Sutherland Equation included in the RCIDCM is defined in the original literature as the Average Sutherland Equation. Average basins being defined as "where the local drainage collection systems for the urban areas within the basin are predominantly storm sewered with curb and gutters, no dry wells or other drainage infiltration areas are known to exist, and the rooftops in the single family areas are not connected to the storm sewer or piped directly to the street curb." The description of average basins does not include discussion related to uses other than single family such as commercial, apartments, industrial, schools, etc. However it is not unusual for uses other than single family to be directly connected or otherwise drain onto impervious surfaces.

The Average Sutherland Equation is:

EIA = 0.1(MIA^1.5) where EIA and MIA are in percent.

A review was made to determine if the Average Sutherland Equation could be considered applicable in the existing developed portions of the basin. While some of the existing development was judged appropriate for this equation, other areas were not. Consequently a decision was made **not to use the Average Sutherland Equation** for the following reasons.

- Current lack of institutional controls in much of the basin to maintain EIA reductions that may exist or otherwise be required to meet the Average Sutherland Equation reduction.

- The study area includes many other land use types than the single family descriptor of the Average Sutherland Equation.

- Various commercial and residential downspouts were observed in the field as discharging onto impervious services, into pipes, or to a very short or steep pervious area.

- Evidence that previous streets without curb and gutter have been or are planned to be reconstructed using curb and gutter and storm sewer systems.

- Steep and/or short pervious areas in much of the basin were assumed to minimize reduction of MIA due to shortness of flow time over pervious area, especially for the less frequent events such as the 100 year storm used as the design basis for a DBDP.

- The Sutherland Equation in the RCIDCM may be overestimating the reduction from MIA to EIA in the study area. As an example, in sub-basin #4, the MIA was estimated from aerial photography as being 32%. Using the Sutherland Equation would result in EIA of 18.1% However, a measurement of the obvious directly connected impervious areas (streets, curb and gutter, curbside sidewalks, and driveways) results in about 16% imperviousness for those directly connected items alone. Essentially the 18% EIA would require all of the approximately 300 homes in this sub-basin to be fully disconnected from the drainage system and that is not the case.

As a substitute method of determining EIA it was judged reasonable **to use the Highly Connected Sutherland Equation** which is:

EIA = 0.4(MIA^1.2) where EIA and MIA are expressed in percent.

This Highly Connected Equation was judged appropriate because:

- It is in line with the statement and concept that “In traditional urban runoff modeling, the EIA for a given basin is usually less than the MIA,”
- Ultimate development will not be of the “highly urbanized” character,
- It accounts for existing development that may have impervious connection more than the definition of average and accounts for the current lack of institutional controls,
- It provides a method to account for initial losses that will occur on impervious surfaces,
- The reduction from MIA to EIA is less than the Average Equation to also account for land uses other than residential.
- It provides some level of safety factor to the values that would otherwise be calculated by the Average Sutherland Equation.

One simple test of reasonableness of the Highly Connected Equation to account for impervious surfaces was reviewed. As noted earlier HMS calculates 100% runoff from impervious areas. By assuming 0.1” is retained on impervious surfaces, this loss by itself could be considered to reduce the MIA by a factor of 3.3% for the 100 Year Design Storm. (0.1” loss/3.06” 100 Year Storm = 3.3%)

The use of Highly Connected Equation is not intended to preclude the City of Rapid City objective of reducing MIA Average Connection conditions or better. Any future reduction in EIA to meet Storm Water Quality Requirements, Low Impact Development concepts, etc., will serve to provide a further safety factor by reducing runoff peaks and volumes.

## 5.6 RUNOFF TRANSFORMATION

The RCIDCM requires that the Snyder Unit Hydrograph method be used for runoff transformation into a hydrograph. For this method, HMS requires input of lag time ($t_p$) and a peaking coefficient ($C_p$).

The RCIDCM equations for the Snyder method are as follows:

$t_p = C_t(LL_c)^{0.3}$

where:

$t_p$ = lag time (hrs)

$C_t$ = lag time coefficient

L = length from the outlet along the main drainage channel (longest flow path) to the drainage divide (miles)

$L_c$ = length from the outlet measured along the main drainage channel to a point perpendicular to the centroid of the drainage basin (miles)

and

$$Q_p = \frac{C_p A}{t_p}$$

where:

$Q_p$ = the peak flow of the unit hydrograph (cfs)

$C_p$ = peaking coefficient

A = the area of the drainage ($mi^2$)

$t_p$ = lag time (hrs)

Length and length to centroid were determined from aerial topographic maps.

The RCIDCM has a range of values for the Lag Time Coefficient ($C_t$) as well as recommended values for a variety of land cover. The following table gives the RCIDCM values for ($C_t$):

**TABLE 2 – RCIDCM VALUES OF LAG TIME COEFFICIENT**

| LAND COVER | LAG TIME COEFFICIENT ($C_t$): | |
|---|---|---|
| | RANGE | RECOMMENDED |
| Mountains, forests, good meadows | 1.8 to 2.2 | 2.00 |
| Range land, pastures, foothills | 0.5 o 1.1 | 0.80 |
| Urban Sewered | 0.3 to 0.9 | 0.60 |

The RCIDCM coefficient values in Table 2 are based simply on values reported in various literature sources and are subject to Engineering judgment when applied to actual studies. No known regional calibration for $C_t$ has been performed. **Because of the judgment and subjectivity associated with the Table 2 $C_t$ values it was judged appropriate <u>for this study</u> to determine Lag Time ($t_p$) based on a method that uses physical parameters and then back calculate to check those results against the RCIDM recommended Snyder coefficients.** This calculation method is an NRCS equation for Watershed Lag which is:

$$L = \frac{(\mathcal{L}^{.8})((S+1)^{.7})}{1900(Y^{0.5})}$$

Where:

L = Lag Time, hours

$\mathcal{L}$ = Flow Length, ft

Y = Average Land Slope, %

S = Maximum Potential Retention, inches, = (1,000/cn')-10, where cn' = retardance factor

Flow length, $\mathcal{L}$, is longest path along which water flows from watershed divide to the outlet.

Average watershed slope, Y, is the average land slope of the watershed, not to be confused with the slope of the flow path. This was calculated using the following NRCS equation:

$Y = 100(CI)/A$

Where:

Y = Average Land Slope, %

C = Sum of length of contour lines that pass through the watershed drainage area, ft.

I = Contour Interval, 10' interval aerial contour lines were used for the summation in each sub basin.

A = Drainage Area, $ft^2$

In regards to Y, no attempt was made to quantify any change in the contours that may occur as future development occurs. It is likely that the watershed land slope will be reduced to some degree as development occurs in the undeveloped portions of the study basin, especially in steeper sub-basins. This would reduce Y which then increases Lag Time to some degree. The increase in lag would be expected to decrease peak flows and as a result provide some margin of additional safety factor to those calculated.

Retardance Factor (cn') is a measure of surface conditions relating to the rate with which runoff concentrates at some point of interest. The retardance factor is approximately the same as the NRCS Curve Number used in their runoff calculation methodology. For the purposes of this study cn' has been assumed to be equal to the Curve Number. Curve numbers were determined using NRCS methods described in NRCS Engineering Manuals.

It should be noted that the above discussions are not meant to serve as a full description or a training document for this NRCS method for determining Lag. Anyone wishing to use this same method must become familiar with the full methodology, applications, and limitations described in the NRCS literature.

A secondary check for reasonableness of Lag Time and $C_t$ was made by using the Snyder Equation with a $C_t$ determined from an equation in USACE EM 1100-2-1417. This equation is also known as the CSU method and was regionally calibrated in the Denver, Colorado area; and as such, it may not be applicable to this study. Nevertheless, it was used as a test of reasonableness due to assumed similarities between this study area and the Denver area. The equation is:

$C_t = 7.81/(I^{.78})$ Where I = Watershed imperviousness in percent

This secondary test used the Snyder Equation for Lag Time with the $C_t$ from this USACE equation.

A table of Lag Time input values and testing data for the above described methods is included in both Appendix A and Appendix B. A review of the table for DBDPA conditions indicates the $C_t$ back calculated from the NRCS Lag Time ranges from 0.44 to 0.90, excluding sub-basins 10 and 17. This range of values fits within the RCIDCM Urban Sewered Range of $C_t$ values (0.3 to 0.9). Sub-basins 8W, 10 and 17 have back calculated $C_t$ values of 1.26, 1.02 and 1.37 respectively; however, these values are judged reasonable given the sub-basin slopes, soil types, and intensity of development.

**Ultimately it was decided to use the results of the NRCS Lag Time equation for the input data for the HMS model <u>in this study</u>** because (1) the NRCS method uses multiple physical measurements rather than pure subjectivity, (2) the method is recognized nationally and some references cite this as perhaps being the most commonly used equation for determining lag time, (3) the back calculations and comparisons to other methods appear reasonable. As such it was determined appropriate to use the NRCS Lag Times in the HMS model.

The RCIDCM states the range of values for the Peaking Coefficient ($C_p$) is from 0.40 to 0.80 (mean of 0.6) with lower values representing less steep slopes and higher numbers for steeper slopes. Due to the lack of any regional $C_p$ calibration or other nationally recognized $C_P$ curves or derivation equations that appeared reasonable for the area, it was judged appropriate to use the RCIDCM values with certain assumptions to set values for $C_P$.

There are noticeable changes in the land slopes in the study area so multiple values of $C_p$ were judged appropriate rather than only using the mean value of 0.6. Using engineering judgment a $C_p$ value of 0.5 was assigned to basins with flatter slopes, judged to be less than between 0.5% and 2.5%. A value of 0.6 was assigned to basins with slopes between 2.5% and 8%. A value of 0.7 was assigned to slopes between 8% and 11%. The land slope was determined from the Average Land Slope (Y) earlier calculated with the NRCS Lag Time. This results in all sub-basins using the mean value of 0.6, except sub-basins 8W, 17 and 18 which use 0.5, and sub-basins 1, 2, and 5 which use 0.7.

## 5.7 SUMMARY RESULTS OF SUB-BASIN HYDROLOGIC ANALYSIS

Summarized results of the Sub-Basin Hydrologic Analysis for the 2 Year, 10 Year, and 100 Year events for existing land use conditions are given on Table 3 at the rear of this chapter.

Summarized results of the Sub-Basin Hydrologic Analysis for the 2 Year, 10 Year, and 100 Year events for future land use conditions are given on Table 4 at the rear of this chapter.

Summarized results that are direct printouts from the HMS run are also included in Appendix A for existing land use and Appendix B for future land use.

Hydrographs of the future land use Sub-basins are included in Appendix C.

It is noted that the sub-basin flows are an approximation simply due to the nature of synthetic design storm analysis, because storms rarely follow ideal patterns, and other factors such as ground cover and infiltration may vary with time or from the assumed conditions used in modeling. The intent of the hydrologic analysis is to provide a reasonably dependable and consistent approximation of runoff.

TABLE 3

PEAK FLOWS FOR EXISTING LAND USE SUB-BASINS

| BASIN NUMBER | DRAINAGE AREA (SQ MI) | DRAINAGE AREA (ACRES) | 2 YEAR PEAK DISCHARGE (CFS) | 10 YEAR PEAK DISCHARGE (CFS) | 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|---|---|---|
| Basin-1 Exist | 0.31 | 198 | 6 | 147 | 387 |
| Basin-2 Exist | 0.08 | 51 | 2 | 62 | 154 |
| Basin 3 Exist | 0.09 | 58 | 11 | 64 | 141 |
| Basin 4 Exist | 0.19 | 122 | 28 | 131 | 280 |
| Basin 5 Exist | 0.25 | 160 | 4 | 101 | 270 |
| Basin 6 Exist | 0.15 | 96 | 14 | 87 | 200 |
| Basin 7 Exist | 0.09 | 58 | 17 | 79 | 163 |
| Basin 8E Exist | 0.04 | 26 | 6 | 29 | 62 |
| Basin 8W Exist | 0.16 | 102 | 9 | 41 | 93 |
| Basin 9 Exist | 0.09 | 58 | 17 | 76 | 158 |
| Basin 10 Exist | 0.14 | 90 | 12 | 58 | 128 |
| Basin 11 Exist | 0.18 | 115 | 10 | 86 | 197 |
| Basin 12 Exist | 0.08 | 51 | 11 | 68 | 141 |
| Basin 13 Exist | 0.17 | 109 | 12 | 78 | 166 |
| Basin14 Exist | 0.17 | 109 | 16 | 72 | 158 |
| Basin 15 Exist | 0.06 | 38 | 8 | 39 | 85 |
| Basin 16 Exist | 0.19 | 122 | 24 | 115 | 244 |
| Basin 17 Exist | 0.09 | 58 | 5 | 28 | 56 |
| Basin 18 Exist | 0.13 | 83 | 9 | 51 | 106 |

TABLE 4

PEAK FLOWS FOR FUTURE LAND USE SUB-BASINS

| BASIN NUMBER | DRAINAGE AREA (SQ MI) | DRAINAGE AREA (ACRES) | 2 YEAR PEAK DISCHARGE (CFS) | 10 YEAR PEAK DISCHARGE (CFS) | 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|---|---|---|
| Basin-1 Future | 0.31 | 198 | 71 | 298 | 612 |
| Basin-2 Future | 0.08 | 51 | 39 | 133 | 246 |
| Basin 3 Future | 0.09 | 58 | 22 | 86 | 170 |
| Basin 4 Future | 0.19 | 122 | 28 | 131 | 280 |
| Basin 5 Future | 0.25 | 160 | 66 | 239 | 472 |
| Basin 6 Future | 0.15 | 96 | 45 | 146 | 282 |
| Basin 7 Future | 0.09 | 58 | 17 | 79 | 163 |
| Basin 8E Future | 0.04 | 26 | 9 | 34 | 70 |
| Basin 8W Future | 0.16 | 102 | 10 | 44 | 97 |
| Basin 9 Future | 0.09 | 58 | 19 | 80 | 165 |
| Basin 10Future | 0.14 | 90 | 14 | 63 | 136 |
| Basin 11 Future | 0.18 | 115 | 33 | 132 | 261 |
| Basin 12 Future | 0.08 | 51 | 40 | 121 | 212 |
| Basin 13 Future | 0.17 | 109 | 20 | 98 | 196 |
| Basin14 Future | 0.17 | 109 | 18 | 77 | 166 |
| Basin 15 Future | 0.06 | 38 | 12 | 46 | 95 |
| Basin 16 Future | 0.19 | 122 | 39 | 144 | 284 |
| Basin 17 Future | 0.09 | 58 | 10 | 38 | 72 |
| Basin 18 Future | 0.13 | 83 | 16 | 69 | 134 |

# 6. HYDRAULICS OVERVIEW

## 6.1 METHODOLOGY

For the purposes of this study Hydraulics is defined as the routing of sub-basin flows through the hydraulic conveyance network. The flows are time delayed as determined by the characteristics of each particular conveyance elements. Flow conveyance elements consist of detention ponds, open channels, and closed conduits.

The selected routing method is Muskingum-Cunge as required by the RCIDCM.

Appendix A contains the hydraulic modeling input data for Existing Land Use Conditions. Appendix B contains the hydraulic modeling input data for the recommended Drainage Basin Design Plan conditions in this report.

Hydraulic calculations and routing includes the following hydraulic elements:

- Sub-basins hydrographs as discussed in Section 5 used for basin runoff.
- Channel and pipe conveyance elements route flow through the system.
- Detention Ponds store and slowly release flows to the downstream system to reduce peaks
- Junctions combine and summarize flows at various locations.

Channel and pipe conveyance elements are discussed in Section 7.

Detention Ponds are discussed in Section 8.

Junctions are discussed in Section 9.

## 6.2 CONVEYANCE ELEMENT ROUTING NETWORK

A Conveyance Element Routing Network was prepared to conceptually represent the storm drainage system as a network of interconnected hydraulic elements.

Figure 12 shows the Existing Condition Hydraulic Routing Network. Figure 13 shows the Existing Condition Hydraulic Routing Network with an Aerial Photo background.

Figure 14 shows the DBDPA Hydraulic Routing Network. Figure 15 shows the DBDPA Hydraulic Routing Network with an Aerial Photo background.

The Routing Network in this study is judged to provide a sufficient number of elements for suitable modeling. The network allows for sub-basin inflow at sub-basin design points and provides flow elements between tributary junctions, between design points, at various road crossings, at detention ponds, and at other locations judged necessary for the model.

The Routing Network was established following major flow patterns. Minor flow systems are beyond the scope of this study.

The Routing Network is different than that used on previous studies due to new and improved base map data, additional sub-basins in this study, previously constructed features, and general needs of this new study.

## 6.3 ROUTING NETWORK NUMBERING SYSTEM

The routing network items use the following numbering system.

- Channels and Pipes: Numbers 1 - 99
- Detention Ponds: 100 Series Numbers
- Direct Flow Elements: 200 Series Numbers

It is noted that Detention Ponds use the same numbers as previous studies. Other routing elements and basins use new numbers established in this study.

## 6.4 SUMMARY RESULTS OF HYDRAULIC ANALYSIS

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for existing land use conditions existing hydraulic conditions are given on Table 5 at the rear this chapter.

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for future land use conditions and the proposed DBDPA hydraulic conditions are given on Table 6 at the rear of this chapter. These values are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

Summarized results that are direct printouts from the HMS run are also included in Appendix A for existing land use and existing hydraulic conditions.

Summarized results that are direct printouts from the HMS run are also included in Appendix B for future land use and future hydraulic conditions. These printouts are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

A summary of the recommended improvements is found in Section 13 of this report and is entitled Major Recommendations Summary, Cost Estimate, and Prioritization

Figures 16, 17, and 18 are enlarged site plan drawings illustrating the areas where major recommendations are proposed.

Hydrographs of the DBDPA condition flow elements, detention ponds, and junctions are included in Appendix C.

Based on a comparison of the existing and DBDPA models the recommendations in this report have the end result of slight increases in peak discharges at the lower end of the study.

***Users of this report need to be aware that the HMS program routes only flows entering the upstream end of the element and ignores the possibility that any adjacent sub-basin flow may be entering the element. Due to this limitation the user must exercise caution when using Model calculated peak channel and pipe flows. Flows for design purposes must be increased appropriately using engineering judgement or other suitable method to account for incoming sub-basin flows.***

## 6.5 FINAL DISCHARGE DISCUSSION

The purpose of this Final Discharge Discussion is to provide a generalized overview of the study results. Additional detailed discussion for various Elements and Detention Ponds follows in the specific report sections.

As evident from the results on Tables 5 and 6 the recommendations in this report result in DBDPA discharges that, depending on location, are either higher or lower than the existing conditions.

Recommendations for Detention Ponds provide adequate controls for areas upstream of Twilight Drive. No major improvements are required upstream of Twilight Drive with the exception of the Detention Pond work and storm sewer on Plateau Lane. A more detailed overview of the Detention Pond System is found in Section 8 of this report.

The Detention Pond recommendations also reduce flows from Twilight Drive downstream to Rapid Creek to manageable levels. Flows at or near Albert Lane are less than existing.

Flows downstream of Albert Lane are above existing rates but are not unreasonable increases. The highest percentage increase is at Junction 211 where the 100 Year flow increases from 944 cfs to 991 cfs or a 5% increase.

Detention Pond 107 as proposed at Reservoir Road and Highway 44 was used to reduce flows in the lower portion of the study area. This pond was judged a reasonable installation because of the density of development that is assumed in that area and because a reasonable location for a regional pond was available.

A review for locations for other Regional Detention Ponds along the main routing network downstream of Reservoir Road was made. For all practical purposes the only remaining undeveloped areas are Sub-basins 11 and 13. In both of these basins the most intense *(higher imperviousness)* development is expected to be near Highway 44 although in Sub-basin 11 there will also be substantial areas of Low Intensity Residential. Based on the review it was judged there are no reasonable locations in Sub-basins 11 and 13 for regional ponds. As a result the increased flows in the downstream portions of the study area are the final recommended flows.

It is noted that improvements recommended later in this report, beginning at Junction 209 and continuing downstream to Junction 213 (Rapid Creek), would still be necessary even if flows were reduced fully to existing condition rates. Because any further reduction in the downstream flows would not noticeably change any recommendations, and because the flows are only slight increases, the final flow increases are judged acceptable.

**However, this does not take away the requirement that future developments in the basins need to construct small onsite ponds to meet the RCIDCM requirement of maintaining runoff to existing conditions prior to implementation of all improvements in this DBDPA. These ponds are also expected to be used to meet Water Quality Capture Volume requirements for stormwater quality treatment.**

TABLE 5

PEAK ELEMENT FLOWS FOR EXISTING LAND USE AND EXISTING HYDRAULIC CONDITIONS

| ELEMENT NUMBER | DRAINAGE AREA (SQ MI) | DRAINAGE AREA (ACRES) | 2 YEAR PEAK DISCHARGE (CFS) | 10 YEAR PEAK DISCHARGE (CFS) | 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|---|---|---|
| 1 | 0.31 | 198 | 3 | 19 | 68 |
| 2 | 0.31 | 198 | 3 | 19 | 68 |
| 3 | 0.39 | 250 | 3 | 38 | 63 |
| 4 | 0.39 | 250 | 3 | 38 | 63 |
| 5 | 0.48 | 307 | 8 | 56 | 89 |
| 6 | 0.48 | 307 | 8 | 56 | 89 |
| 7 | 0.48 | 307 | 8 | 56 | 89 |
| 8 | 0.25 | 160 | 4 | 99 | 268 |
| 9 | 0.4 | 256 | 7 | 20 | 27 |
| 10 | 0.4 | 256 | 7 | 20 | 27 |
| 11 | 0.4 | 256 | 7 | 20 | 27 |
| 12 | 0.49 | 314 | 17 | 78 | 165 |
| 13 | 1.16 | 742 | 45 | 210 | 458 |
| 14 | 1.16 | 742 | 45 | 208 | 457 |
| 15 | 1.2 | 768 | 50 | 235 | 515 |
| 16 | 1.36 | 870 | 54 | 255 | 562 |
| 17 | 1.45 | 928 | 63 | 300 | 665 |
| 17A | 1.45 | 928 | 62 | 295 | 660 |
| 18 | 1.59 | 1018 | 72 | 350 | 779 |
| 19 | 1.59 | 1018 | 71 | 347 | 773 |
| 20 | 1.77 | 1133 | 76 | 409 | 943 |
| 21 | 2.08 | 1331 | 80 | 44 | 1035 |
| 22 | 2.08 | 1331 | 79 | 437 | 1024 |
| 23 | 0.09 | 58 | 17 | 74 | 155 |
| 50 | 0.17 | 109 | 0 | 1 | 2 |
| 51 | 0.23 | 147 | 3 | 11 | 33 |
| 52 | 0.23 | 147 | 3 | 11 | 33 |
| 53 | 0.31 | 198 | 11 | 67 | 140 |
| Pond 100 | 0.4 | 256 | 7 | 20 | 27 |
| Pond 101 | 0.48 | 307 | 8 | 56 | 89 |
| Pond 102 | 0.17 | 109 | 0 | 1 | 2 |
| Pond 103 | 0.31 | 198 | 3 | 19 | 68 |
| Pond 104 | 0.39 | 250 | 3 | 38 | 63 |
| Pond 105 | 0.23 | 147 | 3 | 11 | 34 |
| J201 | 0.39 | 250 | 3 | 67 | 165 |
| J202 | 0.48 | 307 | 13 | 96 | 190 |
| J203 | 0.4 | 256 | 14 | 168 | 435 |
| J204 | 0.49 | 314 | 17 | 79 | 167 |
| J205 | 0.67 | 429 | 28 | 132 | 302 |
| J206 | 1.16 | 742 | 46 | 211 | 458 |
| J207 | 1.2 | 768 | 50 | 236 | 516 |
| J208 | 1.36 | 870 | 55 | 257 | 565 |
| J209 | 1.45 | 928 | 64 | 302 | 648 |
| J210 | 1.59 | 1018 | 73 | 354 | 784 |
| J211 | 1.77 | 1133 | 77 | 410 | 944 |
| J212 | 2.08 | 1331 | 81 | 446 | 1035 |
| J213 | 2.25 | 1440 | 85 | 504 | 1182 |
| J250 | 0.23 | 147 | 8 | 39 | 85 |
| J251 | 0.31 | 198 | 11 | 68 | 142 |
| J260 | 0.19 | 122 | 24 | 115 | 244 |
| J261 | 0.09 | 58 | 5 | 28 | 56 |
| J262 | 0.13 | 83 | 9 | 51 | 106 |

TABLE 6 - PEAK ELEMENT FLOWS FOR DBDPA CONDITIONS
(FUTURE LAND USE AND RECOMMENDED FUTURE HYDRAULIC CONDITIONS)

| ELEMENT NUMBER | DRAINAGE AREA (SQ MI) | DRAINAGE AREA (ACRES) | 2 YEAR DISCHARGE (CFS) | 10 YEAR DISCHARGE (CFS) | 100 YEAR DISCHARGE (CFS) |
|---|---|---|---|---|---|
| 1 | 0.31 | 198 | 2 | 16 | 42 |
| 2 | 0.31 | 198 | 2 | 16 | 42 |
| 3 | 0.39 | 250 | 6 | 31 | 47 |
| 4 | 0.39 | 250 | 6 | 30 | 47 |
| 5 | 0.48 | 307 | 6 | 54 | 101 |
| 6 | 0.48 | 307 | 6 | 53 | 101 |
| 7 | 0.48 | 307 | 6 | 53 | 101 |
| 8 | 0.25 | 160 | 2 | 24 | 71 |
| 9 | 0.4 | 256 | 2 | 8 | 42 |
| 10 | 0.4 | 256 | 2 | 8 | 42 |
| 11 | 0.4 | 256 | 2 | 8 | 42 |
| 12 | 0.49 | 314 | 17 | 77 | 161 |
| 13 | 1.16 | 742 | 45 | 208 | 444 |
| 14 | 1.16 | 742 | 45 | 207 | 439 |
| 15 | 1.2 | 768 | 51 | 236 | 502 |
| 16 | 1.36 | 870 | 57 | 256 | 552 |
| 17 | 1.45 | 928 | 66 | 304 | 668 |
| 17A | 1.45 | 928 | 65 | 304 | 658 |
| 18 | 1.59 | 1018 | 78 | 362 | 782 |
| 19 | 1.59 | 1018 | 78 | 359 | 779 |
| 20 | 1.77 | 1133 | 94 | 448 | 990 |
| 21 | 2.08 | 1331 | 101 | 484 | 1056 |
| 22 | 2.08 | 1331 | 99 | 481 | 1048 |
| 23 | 0.09 | 58 | 19 | 78 | 160 |
| 50 | 0.17 | 109 | 0 | 1 | 26 |
| 51 | 0.23 | 147 | 0 | 9 | 36 |
| 52 | 0.23 | 147 | 0 | 9 | 35 |
| 53 | 0.31 | 198 | 8 | 43 | 80 |
| Pond 100 | 0.4 | 256 | 2 | 8 | 42 |
| Pond 101 | 0.48 | 307 | 6 | 54 | 101 |
| Pond 102 | 0.17 | 109 | 0 | 1 | 26 |
| Pond 103 | 0.31 | 198 | 2 | 16 | 42 |
| Pond 104 | 0.39 | 250 | 6 | 31 | 47 |
| Pond 105 | 0.23 | 147 | 0 | 9 | 36 |
| Pond 106 | 0.25 | 160 | 2 | 24 | 71 |
| Pond 107 | 0.31 | 198 | 8 | 44 | 80 |
| J201 | 0.39 | 250 | 39 | 133 | 247 |
| J202 | 0.48 | 307 | 28 | 110 | 209 |
| J203 | 0.4 | 256 | 45 | 146 | 282 |
| J204 | 0.49 | 314 | 17 | 79 | 164 |
| J205 | 0.67 | 429 | 28 | 132 | 288 |
| J206 | 1.16 | 742 | 45 | 210 | 445 |
| J207 | 1.2 | 768 | 52 | 237 | 503 |
| J208 | 1.36 | 870 | 57 | 261 | 557 |
| J209 | 1.45 | 928 | 67 | 312 | 672 |
| J210 | 1.59 | 1018 | 79 | 365 | 790 |
| J211 | 1.77 | 1133 | 94 | 449 | 991 |
| J212 | 2.08 | 1331 | 102 | 491 | 1070 |
| J213 | 2.25 | 1440 | 111 | 556 | 1231 |
| J250 | 0.23 | 147 | 12 | 46 | 95 |
| J251 | 0.31 | 198 | 40 | 121 | 212 |
| J260 | 0.19 | 122 | 39 | 144 | 284 |
| J261 | 0.09 | 58 | 10 | 38 | 72 |
| J262 | 0.13 | 83 | 16 | 69 | 134 |

# 7. CHANNEL AND PIPE CONVEYANCE ELEMENTS

## 7.1 INTRODUCTION AND MODELING DATA

This section of the report presents discussion for each of the DBDPA Channel and Pipe Conveyance Elements modeled in this report.

Appendix A contains the hydraulic modeling input data for Existing Land Use Conditions.

Appendix B contains the hydraulic modeling input data for the recommended Drainage Basin Design Plan conditions in this report.

Input parameters for channels and pipes consist of slope, roughness, and section geometry.

Input data for slope was taken from as-built or original design drawings where possible. Aerial contours were used for slope where that information was not available. In a few instances, as noted in the detailed discussion of individual elements, field surveys were used to determine pipe slope. Slopes for new or improved channels and pipes are based on the proposed design.

Roughness coefficients are based on engineering judgment. The roughness coefficients were selected to represent conditions as they exist in the field or as recommended. Where necessary, the detailed discussion of individual elements includes a discussion of roughness coefficient.

Section geometry data for pipes consists simply of the pipe diameter. Pipe data was determined from As Builts, field measurements, or recommended improvements. Box culverts were modeled as open top rectangular channels because HMS does not have an input description for closed box culverts.

Simple trapezoidal channel geometry consists of bottom width and side slopes. Channels with composite sections were input as 8 point x and y coordinates. Channel data was determined from as-builts, original designs, aerial contours, engineering judgement based on field observations, or recommended improvements.

It is noted that natural channels, and some manmade channels, vary in shape, slope, and roughness throughout the length of the element. The modeling criteria entered for each channel is considered an average approximation of the particular element.

Unobstructed flow was assumed for all channels and pipes.

Computer Program HY8 was used for analysis of pipe culverts and box culverts where necessary.

Computer Program Flowmaster was used to determine flow depth and velocity for trapezoidal channels using the referenced modeling data and flows.

Computer model UD-Channels was used to determine flow velocity and depth for composite channels using the referenced modeling data and routed flow.

## 7.2 SUMMARY RESULTS

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for existing land use conditions existing hydraulic conditions are given on Table 5 on Page 30 at the rear of Section 6.

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for future land use conditions and the proposed future DBDPA hydraulic conditions are given on Table 6 on Page 31 at the rear of Section 6. These values are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

Summarized results that are direct printouts from the HMS run are also included in Appendix A for existing land use and existing hydraulic conditions.

Summarized results that are direct printouts from the HMS run are also included in Appendix B for future land use and future hydraulic conditions. These printouts are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

A summary of the recommended improvements is found in Section 13 of this report and is entitled Major Recommendations Summary, Cost Estimate, and Prioritization

Figures 16, 17, and 18 are enlarged site plan drawings illustrating the areas where major recommendations are proposed.

Hydrographs of the DBDPA condition flow elements, detention ponds, and junctions are included in Appendix C.

***Users of this report need to be aware that the HMS program routes only flows entering the upstream end of the element and ignores the possibility that any adjacent sub-basin flow may be entering the element. Due to this limitation the user must exercise caution when using Model calculated peak channel and pipe flows. Flows for design purposes must be increased appropriately using engineering judgement or other suitable method to account for incoming sub-basin flows.***

***It is also noted the flow depth and velocity given in the following individual Element discussions are considered approximate because they are based on assumed, idealized, typical channel sections and do not account for any possible backwater.***

## 7.3 DETAILED ELEMENT DISCUSSION

Detailed discussion for each of the Channel and Pipe Elements modeled in the study is included in the section. Each element includes:

- Description of the Element,
- Modeling Information used in the HMS model, and
- Recommendations and Design Flow

Refer to Section 8 for Detention Ponds, Section 9 for Junctions, and Section 10 for modeled “minor” basins.

## ELEMENT 1

**Description:**

Element 1 is the existing 42” RCP outlet pipe draining Detention Pond 103. A flow regulating riser system is located at the upstream end of the pipe for Detention Pond 103.

**Modeling Information:**

The following data was utilized to model the element in HMS.

42” Diameter RCP
n = 0.013
Length = 186’
Slope = 0.0097 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report. The only flow in this pipe is the discharge from Detention Pond 103 which is the same as the routed flow.

| | Element 1- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 16 |
| 100 Year | 42 |
| 100 Year Velocity | 7.3 fps |

**Recommendations:**

The recommended design flows for this pipe are the same as the routed flows above.

No improvements to this outlet pipe are necessary.

Refer to Detention Pond 103 for improvements to the riser system at the upstream end of this pipe.

(End of Element 1 Narrative)

## ELEMENT 2

**Description:**

Element 2 is an existing grass lined channel between Element 1 and Detention Pond 104.

Subdivision master plans in the area indicate the channel will be graded and straightened.

This channel drains into Detention Pond 104.

**Modeling Information:**

The following data was utilized to model the element in HMS.

9' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 1000'
Slope = 0.006 ft/ft

The channel used for modeling is per the information above. The 0.006 ft/ft channel slope is the maximum allowed by the RCIDCM for a grass lined channel. The n value is also per the RCIDCM for capacity check.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 2 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 16 |
| 100 Year | 42 |
| 100 Year Velocity | 2 fps |
| 100 Year Depth | 1.1 ft |

**Recommendations:**

Recommended design flows at the upstream end of the channel are the routed flows given above.

Recommended design flows at the downstream end of the channel need to be increased to account for incoming sub-basin flows. The recommended downstream design flow is from Junction 201 which summarizes Element 2 and Sub-basin 2 flows. The recommended design flow at the downstream end,along with flow depth and velocity, are as follows.

| | Element 2<br>Design Flow at Downstream End<br>(Design Flow = Junction 201 Flow)<br>(cfs) |
|---|---|
| 2 Year | 39 |
| 10 Year | 133 |
| 100 Year | 248 |
| 100 Year Velocity | 4.7 fps |
| 100 Year Depth | 2.7 ft |

Two drop structures, each with about a 3' drop, are estimated as being necessary for a stable channel grade for the segment of channel that will be regraded with the adjacent subdivision.

The segment of the channel within Detention Pond 104 needs to be improved as part of the improvements to that pond. Refer to Detention Pond 104 for those improvements.

Although not modeled in HMS, a low flow or trickle channel should be incorporated into the channel bottom as required by the RCIDCM.

It is recommended that a linear graded maintenance/access road (space) be provided on one side of the channel per the requirements of the RCIDCM.

(End of Element 2 Narrative)

## ELEMENT 3

**Description:**

Element 3 is the existing 30” RCP outlet pipe draining Detention Pond 104. The pipe is under Homestead Street. The existing pipe is simply a culvert with no flow regulating device.

**Modeling Information:**

The following data was utilized to model the element in HMS.

30” Diameter RCP
n = 0.013
Length = 188’
Slope = 0.0071 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report. The only flow in this pipe is the discharge from Detention Pond 104 which is the same as the routed flow.

| | Element 3 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 6 |
| 10 Year | 31 |
| 100 Year | 47 |
| 100 Year Velocity | 10.2 fps |

**Recommendations:**

The recommended design flows for this pipe are the same as the routed flows above.

No improvements to this outlet pipe are necessary.

Refer to Detention Pond 104 for improvements to the riser system at the upstream end of this pipe.

(End of Element 3 Narrative)

## ELEMENT 4

**Description:**

Element 4 is an existing grass lined channel between the Detention Pond 104 and Detention Pond 101. This channel is actually located in the bottom of Detention Pond 101.

**Modeling Information:**

The following data was utilized to model the element in HMS.

65' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.050
Length = 290'
Slope = 0.012 ft/ft

The modeling data is intended to simulate the existing channel.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 4 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 6 |
| 10 Year | 30 |
| 100 Year | 47 |
| 100 Year Velocity | 1.8 fps |
| 100 Year Depth | 0.4 ft |

**Recommendations:**

Recommended design flows at the upstream end of the channel are the routed flows given above. Recommended design flows at the downstream end of the channel need to be increased to account for incoming sub-basin flows. The recommended downstream design flow is from Junction 202 which summarizes Element 4 and Sub-basin 3 flows. The recommended downstream design flow, flow depth and velocity are tabulated below.

| | Element 4<br>Design Flow at Downstream End<br>(Design Flow = Junction 202 Flow)<br>(cfs) |
|---|---|
| 2 Year | 28 |
| 10 Year | 110 |
| 100 Year | 209 |
| 100 Year Velocity | 3.1 fps |
| 100 Year Depth | 1.0 ft |

Improvements are suggested, but are not absolutely necessary. Refer to Detention Pond 101 discussion for a description of the suggested improvements.

(End of Element 4 Narrative)

## ELEMENT 5

**Description:**

Element 5 is the existing 36" RCP outlet pipe system draining Detention Pond 101. The existing pipe is simply a culvert with no flow regulating device at the upstream end.

**Modeling Information:**

The following data was utilized to model the element in HMS.

36" Diameter RCP
n = 0.013
Length = 188'
Slope = 0.0071 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report. The only flow in this pipe is the discharge from Detention Pond 101 which is the same as the routed flow.

| | Element 5 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 6 |
| 10 Year | 54 |
| 100 Year | 101 |
| 100 Year Velocity | 14.3 fps |

**Recommendations:**

The recommended design flows for this pipe are the same as the routed flows above.

No improvements to this outlet pipe are necessary.

Refer to Detention Pond 101 for improvements to the riser system at the upstream end of this pipe.

(End of Element 5 Narrative)

## ELEMENT 6

**Description:**

Element 6 is an existing graded channel between Detention Pond 101 and Plateau Lane. The channel was graded as part of the adjacent subdivision projects. Most of the channel is heavily vegetated with trees and brush.

South Pitch Drive and Avenue A both cross this channel. Both crossings are triple 48" RCP culverts.

**Modeling Information:**

The following data was utilized to model the element in HMS.

25' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.070
Length = 1300'
Slope = 0.007 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 6 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 6 |
| 10 Year | 53 |
| 100 Year | 101 |
| 100 Year Velocity | 2.1 fps |
| 100 Year Depth | 1.5 ft |

**Recommendations:**

The recommended design flows at the upstream end of the channel are the routed flows given above.

Recommended design flows at other locations were determined by linear interpolation of flows between Detention Pond 101 and Junction 205 in order to account for inflows from Sub-basin 4. Interpolated design flows at select locations are given below.

| | Element 6 Interpolated Design Flow At South Pitch Drive (cfs) | Element 6 Interpolated Design Flow At Avenue A (cfs) |
|---|---|---|
| 2 Year | 15 | 20 |
| 10 Year | 50 | 70 |
| 100 Year | 145 | 190 |
| 100 Year Velocity | 2.4 fps | 2.6 fps |
| 100 Year Depth | 1.9 ft | 2.2 ft |

No improvements are required to Element 6 except for routine maintenance as may be determined necessary or requested/performed by adjacent property owners. Much of the channel is experiencing heavy growth in the bottom due to the flat slope and lack of maintenance.

Channel capacity is adequate based on normal depth calculations of the interpolated design flows and the assumed n value being representative of unmaintained or wetland type vegetation conditions. Therefore, it is not necessary to mow or otherwise maintain the channel to short grass. It is recommended that the wetland vegetation be allowed to remain as much as possible to provide water quality improvements. Another reason for allowing the generally unmaintained wetland vegetation to remain is that equipment access to the channel for maintenance will be difficult. Routine maintenance for mowing heavy vegetation, brush removal, etc., can be performed and still maintain the overall "wetland" type channel characteristics.

Both street crossings of Element 6 have capacity to convey the 100 year design flows without overtopping. About 4 feet of freeboard is available at South Pitch Drive and about 1 foot of freeboard is available at Avenue A.

(End of Element 6 Narrative)

## ELEMENT 7

**Description:**

Element 7 is an existing grass lined channel between Element 6 and Element 205. The channel was graded as part of the adjacent subdivision. The channel appearance is generally that of a maintained grass channel.

Plateau Lane is located at the upstream end of this channel. The Plateau Lane crossing is an 8' x 6' reinforced concrete box culvert.

**Modeling Information:**

The following data was utilized to model the element in HMS.

25' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 1100'
Slope = 0.0067 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 7 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 6 |
| 10 Year | 53 |
| 100 Year | 101 |
| 100 Year Velocity | 3.3 fps |
| 100 Year Depth | 1.1 ft |

**Recommendations:**

The recommended design flows at the upstream end of Element 7 are the interpolated flows at Avenue A as described under Element 6 to account for inflows from Sub-basin 4.

Recommended design flows at select locations are given below. Design flows at Plateau Lane were determined using the same linear interpolation as described in Element 6. Design flows at the downstream end of Element 7 are the flows calculated at Junction 205.

| | Element 7 Interpolated Design Flow At Avenue A (cfs) | Element 7 Interpolated Design Flow At Plateau Lane (cfs) | Element 7 Downstream End Design Flow (Junction 205) (cfs) |
|---|---|---|---|
| 2 Year | 20 | 25 | 29 |
| 10 Year | 70 | 85 | 136 |
| 100 Year | 190 | 220 | 292 |
| 100 Year Velocity | 4.0 fps | 4.2 fps | 4.7 fps |
| 100 Year Depth | 1.5 ft | 1.6 ft | 1.9 ft |

No improvements to Element 7 are required. It appears the channel is generally being mowed by adjacent homeowners. Additional channel capacity is available if some level of wetland vegetation or otherwise unmaintained conditions become prevalent.

The Plateau Lane crossing has capacity for the 100 year flow with no overtopping. Approximately 3 feet of freeboard is available.

(End of Element 7 Narrative)

## ELEMENT 8

**Description:**

Element 8 is an existing grass lined channel between Detention Pond 106 and Detention Pond 100. The channel is a graded section from Detention Pond 100 to a location just upstream of Homestead Street. The remaining upper reach of the channel is the natural channel. Steeper portions of the graded channel are lined with TRM or riprap.

Homestead Street and the pedestrian walkway between Patricia Street and Big Sky Drive cross this channel. The Homestead Street crossing consists of 3 -54” RCP culverts. The pedestrian walkway crossing consists of 3 – 60” RCP culverts.

**Modeling Information:**

The following data was utilized to model the element in HMS.

20’ Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.040
Length = 2400’
Slope = 0.013 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all Amendment recommendations in this report.

| | Element 8 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 24 |
| 100 Year | 71 |
| 100 Year Velocity | 3.5 fps |
| 100 Year Depth | 0.9 ft |

**Recommendations:**

The recommended design flows at the upstream end of the channel are the routed flows given above.

Recommended design flows at other locations were determined by linear interpolation of flows between Detention Pond 106 and Junction 203 in order to account for inflows from Sub-basin 6. Interpolated design flows at select locations are given below.

| | Element 8 Interpolated Design Flow At Homestead St. (cfs) | Element 8 Interpolated Design Flow At Pedestrian Crossing (cfs) | Element 8 Downstream Design Flow (Junction 203) (cfs) |
|---|---|---|---|
| 2 Year | 25 | 35 | 45 |
| 10 Year | 80 | 100 | 147 |
| 100 Year | 170 | 210 | 282 |
| 100 Year Velocity | 4.7 fps | 5.0 fps | 5.4 fps |
| 100 Year Depth | 1.4 ft | 1.6 ft | 1.9 ft |

No improvements are required for the channel. Channel capacity is adequate and the velocity in the typical channel is non erosive. However, the channel should be reviewed for stability when undeveloped areas adjacent to the channel are developed. Improvements may be needed pending the outcome of the stability analysis and would be considered as subdivision improvements.

The downstream segment of the channel between the Pedestrian Crossing and Detention Pond 100 is actually in the bottom of Detention Pond 100. Recommended improvements to the bottom of the pond are discussed under Detention Pond 100.

The Homestead Street crossing has capacity to convey the 100 year design flow without roadway overtopping. On the order of 8' of freeboard is available. This crossing was installed in advance of this DBDPA recommendation for new Detention Pond 106. Depending on how the adjacent property develops, it may be possible to modify the inlet end of these pipes to serve as localized detention or for the Post Construction Water Quality Treatment area for that adjacent property.

The Pedestrian crossing has capacity to convey the 100 year design flow without roadway overtopping. Approximately 3' of freeboard is available.

(End of Element 8 Narrative)

## ELEMENT 9

**Description:**

Element 9 is the existing 36" RCP outlet pipe draining Detention Pond 100. A flow regulating riser system is located at the upstream end of the pipe.

**Modeling Information:**

The following data was utilized to model the element in HMS.

36" Diameter RCP
n = 0.013
Length = 290'
Slope = 0.0029 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report. The only flow in this pipe is the discharge from Detention Pond 100 which is the same as the routed flow.

| | Element 9- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 8 |
| 100 Year | 42 |
| 100 Year Velocity | 8.1 fps |

**Recommendations:**

The recommended design flows for this pipe are the same as the routed flows above.

No improvements to this outlet pipe are necessary.

Refer to Detention Pond 100 for improvements to the riser system at the upstream end of this pipe.

(End of Element 9 Narrative)

**ELEMENT 10**

**Description:**

Element 10 is an existing graded channel between Element 9 and Element 11. The downstream end of Element 10 is about 250' south of Avenue A.

Vegetation in the channel is not being maintained and areas of tall grass, brush, and wetland vegetation are present.

The original subdivision plans indicate the channel is lined with TRM upstream of Avenue A.

Avenue A crosses this channel. The crossing consists of twin 54" RCP culverts.

**Modeling Information:**

The following data was utilized to model the element in HMS.

25' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.050
Length = 700'
Slope = 0.022 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 10 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 8 |
| 100 Year | 42 |
| 100 Year Velocity | 2.9 fps |
| 100 Year Depth | 0.6 ft |

**Recommendations:**

The recommended design flows at the upstream end of the channel are the routed flows given above.

Recommended design flows at the downstream end of Element 10 were determined by linear interpolation of flows between Detention Pond 100 and Junction 204 in order to account for inflows from Sub-basin 7. Interpolated design flows are as follows.

| | Element 10 Interpolated Design Flow At Downstream End (cfs) |
|---|---|
| 2 Year | 10 |
| 10 Year | 35 |
| 100 Year | 90 |
| 100 Year Velocity | 3.7 fps |
| 100 Year Depth | 0.9 ft |

Avenue A is only a short distance above the downstream end of Element 10. Therefore, these same flows apply to the Avenue A crossing.

No improvements to Element 10 are necessary. Channel capacity is adequate based on normal depth calculations of the interpolated design flows and the assumed n value being representative of unmaintained conditions. Therefore, it is not necessary to mow or otherwise maintain the channel down to grass lined conditions. It is recommended that wetland vegetation be allowed to remain as much as possible to provide water quality improvements.

The Avenue A crossing has capacity to convey the 100 year design flow without roadway overtopping. Approximately 3' of freeboard is available.

(End of Element 10 Narrative)

## ELEMENT 11

**Description:**

Element 11 is an existing grass lined channel between Element 10 and Junction 204. The channel was graded as part of the adjacent subdivision. The channel appearance is generally that of a maintained grass channel.

Plateau Lane is located towards the downstream end of this channel. The Plateau Lane crossing consists of double 66” Elliptical RCP culverts (83” Span x 53” Rise).

**Modeling Information:**

The following data was utilized to model the element in HMS.

20’ Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 1200’
Slope = 0.005 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report

| | Element 11- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 2 |
| 10 Year | 8 |
| 100 Year | 42 |
| 100 Year Velocity | 2.3 fps |
| 100 Year Depth | 0.8 ft |

**Recommendations:**

Recommended design flow at the upstream end of the channel is the same as the downstream design flow in Element 10.

Recommended design flows at Plateau Lane and the downstream end of Element 11 are the flows calculated at Junction 204.

Recommended design flows at select locations are given below.

| | Element 11 Interpolated Design Flow At Upstream End (cfs) | Element 11 Downstream Design Flow & Plateau Lane (Junction 204) (cfs) |
|---|---|---|
| 2 Year | 10 | 17 |
| 10 Year | 35 | 79 |
| 100 Year | 90 | 164 |
| 100 Year Velocity | 3.0 fps | 3.6 fps |
| 100 Year Depth | 1.2 ft | 1.7 ft |

Improvements consisting of additional storm sewer inlets on Plateau Lane are needed where the street crosses the channel. Additional inlets are needed because storm sewer inlet capacity at the Plateau Lane crossing is inadequate and Sub-basin 7 flows that are on the street will bypass the channel and pipe system and actually leave the County Heights Drainage Basin. Inlet capacity for the 100 year flow should be provided.

Determining the exact number and size of inlets needed at this location requires detailed surveys and design analysis that is beyond the scope of this project. The design analysis will have to include a determination of how much flow turns west from Aurora Drive onto Butte Court, how much flow can be captured at the Avenue A crossing near the upstream end of Element 11, and an analysis of the inlet/storm sewer system on Aurora Drive between Avenue A and Patricia Street.

Based on a cursory examination of the existing inlet system in Sub-basin 7 it is estimated, for the purposes of this report, that a minimum of 8 additional Type E inlets are needed. It is further assumed the storm sewer and inlet system will need to extend from the Plateau Lane crossing of Element 11 to Aurora Drive. Four inlets are assumed to be at the intersection and the remaining 4 inlets placed at the crossing to supplement the existing inlets.

No improvements to the Element 11 channel are required. Additional channel capacity is available if some level of wetland vegetation or otherwise unmaintained conditions become prevalent. The Plateau Lane crossing has capacity for the 100 year flow with no overtopping. Approximately 2 feet of freeboard is available.

(End of Element 11 Narrative)

## ELEMENT 12

**Description:**

Element 12 is an existing grass lined channel between Element 11 and Junction 206. The channel was graded as part of the adjacent subdivision. The channel appearance is generally that of a maintained grass channel.

The downstream end of Element 12 is at the inlet to the Twilight Drive box culvert which is discussed as Element 13.

**Modeling Information:**

The following data was utilized to model the element in HMS.

35' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 900'
Slope = 0.005 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 12- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 17 |
| 10 Year | 77 |
| 100 Year | 161 |
| 100 Year Velocity | 3.2 fps |
| 100 Year Depth | 1.3 ft |

**Recommendations:**

Recommended design flows at the upstream end of the channel are the routed flows given above. These design flows can be used for that portion of the channel that is upstream of the confluence of Elements 7 and 12.

Recommended design flows at the downstream end of Element 12 are the flows calculated at Junction 206. These flows should be used downstream of the confluence of Elements 7 and 12.

| | Element 12 Design Flow At Downstream End (Junction 206) (cfs) |
|---|---|
| 2 Year | 45 |
| 10 Year | 210 |
| 100 Year | 445 |
| 100 Year Velocity | 4.5 fps |
| 100 Year Depth | 2.2 ft |

No improvements to Element 12 are required. It appears the channel is generally being kept in a mowed condition. Additional channel capacity is available in the event some level of wetland vegetation or otherwise unmaintained conditions become prevalent.

(End of Element 12 Narrative)

# ELEMENT 13

**Description:**

Element 13 is the double 12’ x 6’ Concrete Box Culvert under Twilight Drive.

**Modeling Information:**

The following data was utilized to model the element in HMS.

24’ Bottom Rectangular Channel
n = 0.013
Length = 150’
Slope = 0.005 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

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| | Element 13- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 45 |
| 10 Year | 210 |
| 100 Year | 445 |
| 100 Year Velocity | 11.6 fps |

**Recommendations:**

The recommended design flows are the routed flows given above.

No improvements to Element 13 are required.

The box culvert has capacity to convey the 100 year design flow of 445 cfs without overtopping of Twilight Drive. Approximately 18” of freeboard is available.

(End of Element 13 Narrative)

## ELEMENT 14

**Description:**

Element 14 is an existing graded grass lined channel between the Element 13 Twilight Drive box culvert and the Element 15 Albert Lane box culvert.

The channel was constructed by Pennington County circa 1997. A neighboring property owner stated that Pennington County mows the channel. Pennington County owns the drainage lot for the channel between Twilight Drive and Bonnie Lane. The channel is in an easement between Bonnie Lane and Albert Lane.

Original design drawings for the channel show the channel as a trapezoid shape with a 10' bottom, 3:1 slopes, longitudinal grade of 1%, minimum depth of 3.4' and maximum depth of 6.5'. Berms along the edge were used to create channel depth at certain locations.

The channel has 4 vertical gabion drop structures for grade control. The downstream drop structure is immediately upstream of Element 15 and is 2' in height. All other drop structures are 4' in height. The channel is beginning to experience headcutting at the upstream face of the drops.

Leroy Street crosses this channel. The crossing is a 12' x 4' concrete box culvert.

A portion of Roberts Court, a private street located west of the channel, drains to the channel. The street was designed to "buck grade" for this drainage direction. Natural terrain in this area drains to the southwest but the street was graded to drain east to the channel. It is noted this is also one of the locations where the original channel construction had a berm on the west side. Because the street grading was contrary to the terrain, the berm on the west side of the channel was cut through to allow the street to drain into the channel. A limited field survey was performed at this location. The top of berm adjacent to the drainage cutout is at about elevation 3152.5, the channel flow line is about elevation 3149.1, and the Roberts Court drainage pan at the berm cutout is at about 3151.0 or less than 2' above the channel bottom. While not verified by survey it is assumed structures adjacent to this cutout were constructed to at least elevation 3152.5 to be as high as the original berm elevation.

**Modeling Information:**

The following data was utilized to model the element in HMS.

10' Bottom Trapezoid Channel
3:1 Side Slopes
n = 0.035
Length = 1300'
Slope = 0.010 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 14 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 45 |
| 10 Year | 207 |
| 100 Year | 439 |
| 100 Year Velocity | 6.9 fps |
| 100 Year Depth | 3.2ft |

**Recommendations:**

Recommended design flows at the upstream end of the channel are the routed flows given above.

Recommended design flows at the downstream end of the channel are as calculated at Junction 207 to account for the additional inflows from Sub-basin 8E. These design flows are tabulated below.

| | Element 14 Design Flow At Downstream End (Junction 207) (cfs) |
|---|---|
| 2 Year | 52 |
| 10 Year | 237 |
| 100 Year | 503 |
| 100 Year Velocity | 7.2 fps |
| 100 Year Depth | 3.4 ft |

Improvements to the Element 14 channel are recommended in order to provide freeboard. Improvements to the Leroy Street crossing are also recommended.

Based on the original design drawings the Element 14 channel has capacity for the 100 year design flow of 503 cfs; however, no freeboard is available in those areas where the channel was constructed to minimum depth of 3.4 feet. It is recommended the channel be regraded to lower the flow line by a minimum of 1' to provide a minimum of 1' of freeboard.

The proposed 1' freeboard does not include the required additional velocity head depth due to the limited area/retrofit nature of the recommended improvements.

There may be short segments where adequate depth is available; however, for the purposes of this report it is assumed the entire reach of channel needs to be regraded with the exception of the about 200' of channel immediately downstream of Twilight Drive. Based on field observations it is evident that adequate channel depth exists downstream of Twilight Drive for about 200'.

The proposed channel lowering was compared to the original design cross sections. This comparison indicates the lowered channel will fit within the existing 50' drainage easement.

The channel regrading requires each of the drop structures to be modified. The tops of all drops will have to be lowered by a minimum of 1 foot by removing the top row of gabion baskets. It is also necessary to construct armor for the 1 foot drop that is also required near the bottom of the existing drop structures. In some cases it may be possible to move the additional 1 foot bottom drop some distance downstream of the drop as long as the required channel depth can be achieved.

The channel should also be lined with TRM because velocity is approaching 7 fps and because the grade is steeper than the RCIDCM maximum grade criteria of 0.60%.

The existing Leroy Street crossing will overtop. For informational purposes it is estimated the existing condition box culvert has capacity for about 370 cfs before overtopping would begin. Overtopping flows will not return to the channel but rather will overflow to the west away from the channel. It is recommended that capacity of the box culvert be increased by replacing the precast zero degree wing walls with a 90 degree headwall with 1.5 to 1 bevels.

Leroy Street should also be reconstructed to provide an overtopping section that returns flow to the downstream channel. This will require the new roadway surface be only slightly above the box culvert. For the purposes of this report it has been assumed the overtopping section is 40' wide and consists of reinforced concrete pavement placed directly on top of box culvert.

Using 480 cfs as the design flow at Leroy Street, with the above recommendations, results in about 455 cfs through the box and about 25 cfs overtopping. The overtopping flow will be on the order of 6" deep.

A limited field survey was made for the analysis of this box culvert. The flow line out is 3157.0, flow line in is 3157.44, and the length is 40'. The existing overtopping elevation is about 3163.1. The proposed new overtopping section is at elevation 3162.75 and is 40' in length. With this overtopping section and the proposed headwall, the 100 year water elevation was calculated by HY8 as 3163.1

(End of Element 14 Narrative)

## ELEMENT 15

**Description:**

Element 15 is the box culvert located under Albert Lane.

The box culvert is 10’ x 4’ and is about 234’ in length. The culvert was constructed by Pennington County circa 1997. The box culvert is located in easements on both sides of Albert Lane.

The inlet end of the box culvert is about 120’ upstream of Albert Lane. The easement granted to the County for structure through this property states it must be a covered structure and not an open channel through the property.

The box culvert extends about 110’ downstream of Albert Lane.

Houses are very close to both sides of the culvert both upstream and downstream of Albert Lane. There is no overflow section between the houses or at Albert Lane. Any overtopping flows would travel in a southwest direction.

**Modeling Information:**

The following data was utilized to model the element in HMS.

10’ Bottom Rectangular Channel
n = 0.013
Length = 234’
Slope = 0.0043 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 15- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 51 |
| 10 Year | 237 |
| 100 Year | 503 |
| 100 Year Velocity | 12.6 fps |

**Recommendations:**

Recommended design flows are the flows given above.

Improvements to Element 15 are recommended.

The existing box culvert does not have capacity to carry the 100 year design flow of 503 cfs without overtopping. Overtopping flows may flood nearby structures due to the lack of any defined overflow section. It is recommended the box culvert inlet be modified to allow the 100 year flow to be conveyed in the box.

For informational purposes it is noted the existing condition box culvert has capacity for about 445 cfs before overtopping would begin at elevation 3151.20.

Grading and street reconstruction to provide an overflow route over the box culvert route was judged impractical because of culvert and structure elevations. The top of the box culvert on the south side of the street is at essentially the same elevation as the adjacent garage to the west. This garage is also lower than the roadway. The house on the west side of the box culvert upstream of the roadway is only about 1’ above the top of the box culvert. The roadway would also have to be reconstructed to create an overtopping sag above the culvert.

The recommended improvement consists of replacing the existing flared wingwall inlet with a new inlet. The proposed inlet consists of a 90 degree headwall with 1.5 to 1 bevels. It is also necessary to increase the headwall height by about 10” above existing and carry this increased elevation upstream as necessary with walls or grading.

A limited field survey was made for the analysis of this box culvert. The flow line out is 3142.67, flow line in is 3143.74, and the top of existing headwall elevation is 3151.20. The proposed new top of headwall elevation is 3152.0. With the improved inlet as described, the 100 year water elevation was calculated by HY8 as 3151.4; thus, the raised headwall provides about 6” of freeboard against overtopping. An n value of 0.013 was used for the HY8 analysis.

Upstream berm elevations, and presumably adjacent structures, near Roberts Court are at elevation 3152.5, as discussed in Element 14. Thus approximately 1 foot of freeboard is provided for these structures as compared to the estimated 100 year water elevation noted in the previous paragraph.

Although a limited survey was performed, it is still necessary to perform a detailed survey during final design to verify and or adjust the design and freeboard requirements as necessary to fit conditions. The detailed survey may also lead to the conclusion that small overtopping flows would be allowable which may provide additional freeboard for the upstream structures. In any case, it is judged that the box culvert inlet improvements are still necessary.

The width of the current easements does not allow another pipe to be installed parallel to the box culvert. Complete removal and replacement of the box culvert with a larger box was judged to be cost prohibitive. Shortening the box culvert to reduce headloss through the long barrel by using a concrete channel through the adjoining properties is not option due to the easement stipulations.

An option to create an overflow path would be to purchase the properties on the west side of the channel on each side of Albert Lane. An overflow path can be created after demolition of the structures on those properties. The lots are large enough that they could then be sold for redevelopment with the new structures away from the overflow. This option would require further investigation in the future due to property purchase issues.

(End of Element 15 Narrative)

## ELEMENT 16

**Description:**

Element 16 is an existing graded channel between the Element 15 Albert Lane box culvert and Junction 209 which is where the storm sewer from Sub-basin 9 enters the system. The channel was constructed by Pennington County circa 1997.

A 2' high gabion drop structure is located about 75' downstream of the Albert Lane box culvert.

It is apparent from review of original design documents that the channel was intended to function as a grass lined channel with corresponding roughness value for grass. However; towards the lower reach of the channel, the bottom has developed wetland type vegetation such as cattails, marsh grasses, etc. Woody or shrub type wetland vegetation does not appear to be present. The side slopes are grass. Based on field observations, it appears the channel bottom and side slopes are being mowed at least in the spring and fall. It is unknown who is doing the mowing.

A trickle flow was observed in the channel during the growing season. The trickle flow appears to be from leakage at the waste gate on the Hawthorne Irrigation Ditch.

Original design drawings show the channel as a trapezoid shape with a 10' bottom, 4:1 slopes, and longitudinal grade of 0.70%. Minimum depth of the designed channel was 4'.

Based on interpolation of the GIS aerial contours the available minimum depth is about 5'. This 5' depth area begins more or less with Shad Street at Mercury Drive and extends about 800 feet upstream. The remainder of the channel appears to have at least 6' of available depth. Berms have been used along much of the channel to create these existing depths.

**Modeling Information:**

The following data was utilized to model the element in HMS.

10' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 2190'
Slope = 0.007 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 16 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 57 |
| 10 Year | 256 |
| 100 Year | 552 |
| 100 Year Velocity | 6.1 fps |
| 100 Year Depth | 3.7 ft |

**Recommendations:**

The recommended design flows at the upstream end of the channel are the routed flow given above.

About 30% of Sub-Basin 10 drains into this channel upstream of Junction 209. The Element 16 design flow was thus calculated by adding 30% of the Sub-basin 10 flow to the Element 16 flow. The resulting design flow for the downstream end of Element 16 is tabulated below.

| | Element 16 Design Flow At Downstream End (cfs) |
|---|---|
| 2 Year | 65 |
| 10 Year | 280 |
| 100 Year | 600 |
| 100 Year Velocity | 6.2 fps |
| 100 Year Depth | 3.8 ft |

Recommended improvements to the Element 16 channel consist of repairing the leaking waste gate on the Hawthorne Irrigation Ditch. With the elimination of the trickle flow from the waste gate it is assumed that the channel can be mowed and maintained as a grass lined channel with an n value of 0.035.

Under these conditions the maximum estimate flow depth is about 3.8' compared to the interpolated maximum channel depth of 5'. This then provides about 1.2' of freeboard.

The proposed 1.2' freeboard does not include the required additional velocity head depth due existing nature of the channel. It is noted that the channel has capacity for nearly 1,100 cfs if flowing 5' deep.

In the interim it is important that the wetland vegetation in the channel continue to be mowed or otherwise maintained until such time as the channel becomes grass lined. Under the assumption of an n value of 0.044, the normal depth would be about 4.2' which would contain the flows to the channel but with less than desired freeboard. A n value of 0.044 can generally be considered as (1) a channel with NRCS Retardance Class B vegetation which can generally be described as various grasses on the order of 1 to 2' in height, or (2) an earthen winding sluggish channel with dense weeds or aquatic growth in the bottom, or (3) a channel generally described as having a smooth degree of irregularity, gradual to no variation in cross section, no obstructions, medium vegetation effects, and only minor meandering.

It is also recommended that a linear graded maintenance/access road (space) and easement be provided on one side of the channel per the requirements of the RCIDCM during the platting of the remaining undeveloped property along the channel.

(End of Element 16 Narrative)

## ELEMENT 17

**Description:**

Element 17 is an existing graded channel between Junction 209 and Reservoir Road.

Element 17 includes the box culvert under Reservoir Road. The channel and box culvert were constructed by Pennington County circa 1997. A 3' high gabion drop structure is located just upstream of the Reservoir Road box culvert.

It is apparent from review of original design documents that the channel was intended to function as a grass lined channel with corresponding roughness value for grass. However; the channel bottom has developed wetland type vegetation consisting of cattails, marsh grasses, etc. Woody or shrub type wetland vegetation does not appear to be present. The side slopes are grass. Based on field observations, it appears the channel bottom and side slopes are being mowed at least in the spring and fall. It is unknown who is doing the mowing.

A trickle/base flow was observed in the channel throughout the growing season. It is appears much of the trickle/base flow is from the "leaking" structures on the Hawthorne Irrigation Ditch. One of these structures is the waste gate discussed under Element 16. The other "leak" is at the Hawthorne Ditch overflow/waste gate at Reservoir Road. That structure was constructed as part of the Reservoir Road reconstruction project in 2011. It is also possible that high groundwater, basement sump pumps, roadway under drains, or utility under drains may be contributing to the trickle/base flow.

The original design drawings show the channel as being a trapezoid shape with a 10' bottom, side slopes of 3:1 and 4:1, and a longitudinal grade of 0.50%. Minimum depth of the designed channel was 4'.

Based on interpolation of the GIS aerial contours, plus a limited number of survey points, the available minimum channel depth appears to be about 4' near the easement on the west side of the channel line. Adjacent homes appear to be elevated to some degree above the channel.

**Modeling Information:**

The following data was utilized to model the improved element in HMS.

8 Point Composite Channel
Low Flow Channel: 10' Wide Bottom, 2' Deep, 3:1 Slopes
Low Flow Channel n = 0.043
Overbanks: 8' Bottom Each Side of Low Flow Channel, 3.5' Deep, 3:1 Slopes
Overbank n = 0.035
Length = 750'
Slope = 0.005 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts a typical composite low flow channel n of 0.065 down to 0.043 to account for overall flow depth in the entire channel.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 17 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 66 |
| 10 Year | 304 |
| 100 Year | 668 |
| 100 Year Velocity | 5.3 fps |
| 100 Year Depth | 4.3 |

**Recommendations:**

Element 17 carries the flow calculated at Junction 209 plus flow from about 40% of Sub-basin 10. The recommended Element 17 design flow was thus calculated by adding 40% of the Sub-basin 10 flow to the Junction 209 flow. Design flows for Element 17 are tabulated below.

| | Element 17 Design Flow (cfs) |
|---|---|
| 2 Year | 75 |
| 10 Year | 340 |
| 100 Year | 730 |
| 100 Year Velocity | 5.3 fps |
| 100 Year Depth | 4.4 ft |

It is recommended the channel be improved with a composite shape for capacity and water quality purposes. The composite shape should be like the HMS modeling data or some other satisfactory configuration. The existing channel does not have capacity to carry the design flow even if the n value is reduced to that of grass lined channel.

It is believed the low flow portion of the composite channel will take on characteristics of a wetland channel even if the leaking irrigation structures are repaired. It is possible trickle flows may still enter the channel from high groundwater, underdrains, and sump pumps. The channel is also far enough down the basin that trickle flows, as are common in most urban basins, from various sources such lawn water, car washing, etc., may become common.

Reconstruction will require lowering of the channel bottom about 18" which is possible by lowering the top of the existing drop structure located upstream of the Reservoir Road box culvert. The channel should be lowered enough to provide a minimum of 5.5' of channel depth.

A new drop structure will be necessary at the upstream end of lowered Element 17 to transition back to Element 16. An option may be to regrade the Element 16 channel for some distance upstream for grade transition.

The existing easement is 60' feet in width and the proposed composite section, at 5.5 feet total depth requires a 59' top width. Based on a review of the original design cross sections, and a limited number of survey shots, the proposed channel will theoretically fit into the existing easement. However, it is possible that grading outside of the easement into the Reservoir Road right of way may be necessary or some additional temporary slope easements may be needed on the adjacent lots.

The recommended improvements provide 1.1’ of freeboard. This does not include the full velocity head depth due to the limited area/retrofit nature of the recommended improvements. It is also noted that at 5.5’ deep the channel would carry 1,340 cfs or nearly twice the design flow.

The composite channel will simplify the maintenance requirements of the channel. The low flow wetland bottom channel will not need regular mowing, rather the vegetation can remain similar to a channel n value of 0.065. The overbanks will need to be maintained to characteristics of a grass lined channel.

The wetland vegetated low flow channel is conducive to water quality enhancement. The proposed low flow channel, based on shallow flow n value of 0.065, has capacity for approximately 65 cfs or about 85% of the 2 year storm. The 2 year storm velocity in the low flow channel is about 2 fps.

A limited field survey was made for the analysis of the Reservoir Road 12’ x 6’ box culvert. The flow line out is 3118.15, flow line in is 3118.65 the overtopping elevation of the roadway sag north of the box is 3127.9. No improvements to the box culvert are necessary. As calculated by HY8 the box culvert will convey the 100 year design storm but is on the verge of overtopping. No improvements to culvert are necessary.

As noted the existing channel does not have capacity for the design flows. Normal depth for the existing trapezoid channel, based on the original design section, a maintained wetland n value of 0.044, and assuming all flow is contained in the trapezoid section is 5.2’. If the n value were reduced to 0.035 the normal depth would be 4.7’ which still exceeds the estimated minimum available depth at the channel. It is important that the channel be maintained to keep the vegetation as short as possible prior to the channel being improved.

(End of Element 17 Narrative)

## ELEMENT 17A

**Description:**

Element 17A is an existing graded channel between Elements 17 and 18.

The channel begins at Reservoir Road and ends at Longview Road. Element 17A includes the box culvert under Longview Road.

The channel has a 4’ high gabion drop structure just upstream of Longview Road and a 2’ gabion drop structure about 150 downstream of Reservoir Road. The channel and box culvert were constructed by Pennington County circa 1997.

It is apparent from review of original design documents that the channel was intended to function as a grass lined channel with corresponding roughness value for grass. However; the channel bottom has developed wetland type vegetation consisting of cattails, marsh grasses, etc. Woody or shrub type wetland vegetation does not appear to be present. The side slopes are grass. Based on field observations, it appears the channel bottom and side slopes are being mowed at least in the spring and fall. It is unknown who is doing the mowing.

A trickle/base flow was observed in the channel throughout the growing season. Same as with Element 17, the majority of the trickle/base flow appears to be from “leaking” structures on the Hawthorne Irrigation Ditch. It is also possible that high groundwater, basement sump pumps, roadway under drains, or utility under drains may be contributing to the trickle/base flow.

Original design drawings indicate the channel as a trapezoid shape with a 10’ bottom, 4:1 side slopes and longitudinal grade of 0.65%. Minimum depth of the designed channel was 4.5’. Berms were used to create the channel depth at certain locations.

Based on information in the original design drawings and interpolation of the GIS aerial contours the available minimum channel depth appears to be about 4’ to 4.5’. Adjacent homes appear to be elevated to some degree above the channel.

**Modeling Information:**

The following data was utilized to model the improved channel in HMS.

8 Point Composite Channel
Low Flow Channel: 10’ Wide Bottom, 2’ Deep, 3:1 Slopes
Low Flow Channel n = 0.045
Overbanks: 14’ Bottom Each Side of Low Flow Channel, 3.5’ Deep, 4:1 slopes
Overbank n = 0.035
Length = 1000’
Slope = 0.0065 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts a typical low flow channel n of 0.065 down to 0.045 to account for the overall depth of flow in the entire channel.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 17A - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 65 |
| 10 Year | 304 |
| 100 Year | 658 |
| 100 Year Velocity | 5.0 fps |
| 100 Year Depth | 3.8 ft |

**Recommendations:**

Recommended design flows at the upstream end are the same as the design flow calculated for Element 17.

Recommended design flows at the downstream end of Element 17A is the flow calculated at Junction 210. Design flows for Element 17A are tabulated below.

| | Element 17A Upstream Design Flow (cfs) (Same as Element 17) | Element 17A Downstream Design Flow (cfs) (From Junction 210) |
|---|---|---|
| 2 Year | 75 | 79 |
| 10 Year | 340 | 365 |
| 100 Year | 730 | 790 |
| 100 Year Velocity | 5.2 fps | 5.4 fps |
| 100 Year Depth | 3.9 ft | 4.0 ft |

It is recommended the channel be improved with a composite shape for capacity and water quality purposes. The composite shape should be like the HMS modeling data or some other satisfactory configuration.

The existing channel does not have capacity to carry the design flow even if the n value is reduced to that of grass lined channel. It is assumed that future development will desire a confined channel area rather than allowing flooding of the broad area that will occur with existing conditions.

It is believed the low flow portion of the composite channel will take on characteristics of a wetland channel even if the leaking irrigation structures are repaired. It is possible trickle flows may still enter the channel from high groundwater, underdrains, and sump pumps. The channel is also far enough down the basin that trickle flows, as are common in most urban basins, from various sources such lawn water, car washing, etc., will be prevalent.

Reconstruction will require lowering of the channel bottom about 18” which is possible by lowering the top of the existing drop structure located upstream of the Longview Road box culvert. The channel needs to be lowered enough to provide a minimum of 5.5’ of channel depth.

A new drop structure will be necessary near the downstream end of the Reservoir Road box culvert as a result of the lowered channel.

The proposed channel depth of 5.5’ provides about 1.5’ of freeboard for the 100 year design flows (1’ + velocity head+-).

The existing easement is 60' feet in width. At the proposed total depth of 5.5' the proposed composite channel top width is 80' which requires additional easement.

An alternate overbank bottom width of 8' on each side of the low flow channel, plus 3 to 1 side slopes for the overbank areas, would theoretically fit within the existing easement; however, channel flow depth increases to 4.3'.

The composite channel will simplify the maintenance requirements of the channel. The low flow wetland bottom channel will not need regular mowing, rather the vegetation can remain similar to a channel n value of 0.065. The overbanks will need to be maintained to the characteristics of a grass lined channel.

The wetland vegetated low flow channel is conducive to water quality enhancement. The proposed low flow channel, based on shallow flow n value of 0.065, has capacity for approximately 75 cfs or nearly all of the 2 year storm. The 2 year storm velocity in the low flow channel is about 2.6 fps. Additional discussion related to water quality treatment in improved channels is found in Section 12.

A maintenance/access "road" should be provided along the channel per the requirements of the RCIDCM. This will require additional easement width.

If future development will provide a wider drainage route an option to be considered would be to modify the existing low flow channel without lowering the bottom, provide a wider overbank area to lessen required flow depth, and grade the adjacent property to be above the channel as necessary. Detailed survey is needed to determine if this option is feasible. This option still follows recommendation for a composite channel shape.

Improvements to overtopping for the Longview Road box culvert are also recommended. The Longview Road crossing is a 10' x 6' box culvert. Longview Road will overtop during the 100 year event. Based on information from the original design drawings *(converted to 1929 vertical datum in this report by subtracting 1.6' from those plans which were prepared on 1988 vertical datum)* the flow line out is 3103.76, flow line in is 3104.21, and the existing overtopping elevation west of the box is at 3114.7. The existing overtopping location is about 120' west of the culvert and will overtop onto private property. It is recommended that the overtopping location be moved near the culvert so overtopping flow will return to the channel within the existing drainage easement. For the purposes of this analysis it was assumed the new overtopping elevation would be a 50' wide section at elevation 3114.0.

Based on the revised overtopping data, HY8 calculations indicate the Longview Road box culvert will pass 645 cfs prior to overtopping. During the 100 year event about 710 cfs will flow through the box and about 80 cfs will overtop to a depth of about 6".

As noted the existing channel does not have capacity for the design flows. Normal depth for the existing trapezoid channel, based on the original design section, a maintained wetland n value of 0.044, and assuming all flow is contained to the trapezoid section is 4.9'. If the n value were reduced to 0.035 the normal depth would be 4.4' which still exceeds the estimated minimum available depth at the channel. It is important that the channel be maintained to keep the vegetation as short as possible for reduced flow depth prior to the channel being improved.

(End of Element 17A Narrative)

## ELEMENT 18

**Description:**

Element 18 is an existing graded channel between Elements 17A and 19. The channel is located between Longview Road and the Murphy Irrigation Ditch.

The channel includes a 3’ high gabion drop structure about 300’ upstream of the irrigation ditch and another 5’ high gabion drop structure about 650’ upstream of the irrigation ditch. The channel was constructed by Pennington County circa 1997.

It is apparent from review of original design documents that the channel was intended to function as a grass lined channel with corresponding roughness value for grass. However; the channel bottom has developed wetland type vegetation consisting of cattails, marsh grasses, etc. Woody or shrub type wetland vegetation does not appear to be present. The side slopes are grass. Based on field observations, it appears the channel bottom and side slopes are being mowed at least in the spring and fall.

A trickle/base flow was observed in the channel throughout the growing season. Same as with Elements 17 and 17A, the majority of the trickle/base flow appears to be from “leaking” structures on the Hawthorne Irrigation Ditch. It is also possible that high groundwater, basement sump pumps, roadway under drains, or utility under drains may be contributing to the trickle/base flow.

Original design drawings indicate the channel as a trapezoid shape with a 10’ bottom, 4:1 side slopes and longitudinal grade of 0.70%. Minimum depth of the designed channel was 4.5’. Berms were used to provide flow depth along part of the channel.

Based on information in the original design drawings and interpolation of the GIS aerial contours the available minimum channel depth appears to be about 4’ to 4.5’. Nearby homes appear to be elevated to some degree above the channel.

**Modeling Information:**

The following data was utilized to model the improved element in HMS.

8 Point Composite Channel
Low Flow Channel: 10’ Wide Bottom , 2’ Deep, 3:1 Slopes
Low Flow Channel n = 0.045
Overbanks: 15’ Bottom Each Side of Low Flow Channel, 3.5’ Deep, 4:1 Slopes
Overbank n = 0.035
Length = 1650’
Slope = 0.0070 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts a typical low flow channel n of 0.065 down to 0.045 to account for the overall depth of flow in the entire channel.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 18 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 65 |
| 10 Year | 362 |
| 100 Year | 782 |
| 100 Year Velocity | 5.4 fps |
| 100 Year Depth | 3.9 ft |

**Recommendations:**

Recommended design flows at the upstream end of Element 18 are the same at the routed flows above.

Element 18 carries the flow calculated at Junction 210 plus additional flow from Sub-basin 11. Recommended design flow at the downstream end of Element 18 was thus determined by linear interpolation between Junctions 210 and 211. Design flows at the downstream end of Element 18 are tabulated below.

| | Element 18 Downstream Design Flow (cfs) |
|---|---|
| 2 Year | 90 |
| 10 Year | 410 |
| 100 Year | 900 |
| 100 Year Velocity | 5.7 fps |
| 100 Year Depth | 4.1 ft |

It is recommended the channel be improved with a composite shape for capacity and water quality purposes. The composite shape should be like the HMS modeling data or some other satisfactory configuration.

The existing channel does not have capacity to carry the design flow even if the n value is reduced to that of grass lined channel. It is assumed that future development will desire a confined channel area rather than allowing flooding of the broad area that will occur with existing conditions.

It is believed the low flow portion of the composite channel will take on characteristics of a wetland channel even if the leaking irrigation structures are repaired. It is possible trickle flows may still enter the channel from high groundwater, underdrains, and sump pumps. The channel is also far enough down the basin that trickle flows, as are common in most urban basins, from various sources such lawn water, car washing, etc., may become prevalent.

Reconstruction will require lowering the channel bottom to provide total channel depth of 5.5 feet. Lowering the channel is made possible by removal of drop structure height as necessary. Lowering the channel by 3.5’ is expected upstream of the 5’ drop structure because the current channel includes a 2’+- high berm along most of its length. The new channel will eliminate this berm so that positive drainage from the adjacent property to the channel is possible.

In addition to the lowering, the drop structures will also have to be widened to fit the new channel shape. A new drop structure will be necessary at the downstream end of the Longview Road box culvert as a result of the lowered channel.

The proposed channel depth of 5.5' provides about 1.4' of freeboard for the 100 year design flows (1' + velocity head+-).

The existing easement is 60' feet in width. At the proposed total depth of 5.5' the proposed composite channel top width is 80' which requires additional easement.

An alternate overbank bottom width of 8' of each side of the low flow channel, plus 3 to 1 side slopes for the overbank areas, would theoretically fit within the existing easement; however, channel flow depth increases to 4.3'.

The composite channel will simplify the maintenance requirements of the channel. The low flow wetland bottom channel will not need regular mowing, rather the vegetation can remain similar to a channel n value of 0.065. The overbanks will need to be maintained to the characteristics of a grass lined channel.

The wetland vegetated low flow channel is conducive to water quality enhancement. The proposed low flow channel, based on shallow flow n value of 0.065, has capacity for approximately 77 cfs or about 85% of the 2 year storm. The 2 year storm velocity in the low flow channel is about 2.4 fps. Additional discussion related to water quality treatment in improved channels is found in Section 12.

A maintenance/access "road" should be provided along the channel per the requirements of the RCIDCM. This will require additional easement width.

The Murphy Irrigation ditch is at the downstream end of this channel. An existing inverted irrigation siphon, constructed with the County project, carries the irrigation flow under the channel. Based on the original design drawings approximately 5' of channel depth exists at this location. The design drawings indicate the siphon has about 37 feet of "flat" pipe under the existing 10' wide channel. It will be necessary to warp the proposed composite section to a section that fits the over the siphon. The channel will not be lowered at this location.

If future development will provide a wider drainage route an option to be considered would be to modify the existing low flow channel without lowering the bottom, provide a wider overbank area to lessen required flow depth, and grade the adjacent property to be above the channel as necessary. Detailed survey is needed to determine if this option is feasible. This option still follows recommendation for a composite channel shape.

As noted the existing channel does not have capacity for the design flows. Normal depth for the existing channel, based on the original design information, a maintained wetland n value of 0.044, and assuming all flow is contained to the trapezoid section is 5.1'. If the n value were reduced to 0.035 the normal depth would be 4.6' which still exceeds the estimated minimum available depth at the channel. There are no existing homes immediately adjacent to the channel, nevertheless, it is recommended the channel be maintained to keep the vegetation as short as possible for reduced flow depth prior to the channel being improved.

(End of Element 18 Narrative)

## ELEMENT 19

**Description:**

Element 19 is an existing graded channel between Elements 18 and Junction 211. The channel is located between the Murphy Irrigation Ditch and Highway 44.

The existing channel has a 3' high gabion drop structure about 100' upstream of Highway 44 and a 5' drop structure about 450' downstream of the irrigation ditch. Most of the channel was constructed by Pennington County circa 1997. The extreme downstream end of the channel, including the 3' drop structure, was constructed by SDDOT in 2008 as part of the Highway 44 reconstruction project.

It is apparent from review of original design documents that the channel was intended to function as a grass lined channel with corresponding roughness value for grass. However; the channel bottom has developed wetland type vegetation consisting of cattails, marsh grasses, etc. Some woody or shrub type wetland vegetation is beginning to appear. The side slopes are grass.

A trickle/base flow was observed in the channel throughout the growing season. A base flow was also observed at certain times outside of the growing season. Same as with upstream channels, the majority of the trickle/base flow appears to be from "leaking" structures on the Hawthorne Irrigation Ditch. However, it is believed that high groundwater, utility underdrains, and sump pumps may be contributing to the trickle/base flow as evidenced by the presence of these flows outside of the irrigation season.

Original design drawings indicate the channel as a trapezoid shape with a 10' bottom, 4:1 side slopes and longitudinal grade of 0.70%. Minimum depth of the designed channel was 4.5'. Berms were used to create channel depth along parts of the channel.

Based on information in the original design drawings and interpolation of the GIS aerial contours the available minimum channel depth appears to be about 4' to 4.5'.

**Modeling Information:**

The following data was utilized to model the improved element in HMS.

8 Point Composite Channel
Low Flow Channel: 14' Wide Bottom, 2' Deep, 3:1 Slopes
Low Flow Channel n = 0.045
Overbanks: 15' Bottom Each Side of Low Flow Channel, 3.5' Deep, 4:1 Slopes
Overbank n = 0.035
Length = 1400'
Slope = 0.0070 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts a typical low flow channel n of 0.065 down to 0.045 to account for overall depth of flow in the entire channel.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all Amendment recommendations in this report.

| | Element 19 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 77 |
| 10 Year | 359 |
| 100 Year | 779 |
| 100 Year Velocity | 5.3 fps |
| 100 Year Depth | 3.8 ft |

**Recommendations:**

Recommended design flow at the upstream end of Element 19 is the same as the design flow calculated for Element 18.

Recommended design flow at the downstream end of Element 19 is the flow calculated at Junction 211. Design flows for Element 19 are tabulated below.

| | Element 19 Upstream Design Flow (cfs) (Same as Element 18) | Element 19 Downstream Design Flow (cfs) (From Junction 211) |
|---|---|---|
| 2 Year | 90 | 94 |
| 10 Year | 410 | 449 |
| 100 Year | 900 | 991 |
| 100 Year Velocity | 5.6 fps | 5.8 fps |
| 100 Year Depth | 4.0 ft | 4.1 ft |

It is noted that about 10% of the Sub-basin 11 flows would not actually reach Element 19, rather they are on the south side of the highway or cross the highway via culverts to the east. However; for the purposes of Element 19, it has been assumed all of Sub-basin 11 contributes flow to the channel. This assumption is well within the accuracy of the modeling and calculation. It is also possible in the future that some of the Sub-basin 11 flows on the north side of the highway will be redirected to the channel rather than the secondary culverts under Highway 44.

It is recommended the channel be improved with a composite shape for capacity and water quality purposes. The composite shape should be like the HMS modeling data or some other satisfactory configuration.

The existing channel does not have capacity to carry the design flow even if the n value is reduced to that of grass lined channel. It is assumed that future development will desire a confined channel area rather than allowing broad flooding that will occur under existing conditions. Therefore an improved channel is recommended.

It is believed the low flow portion of the composite channel will take on characteristics of a wetland channel even if the leaking irrigation structures are repaired. It is possible trickle flows may still enter the channel from high groundwater, underdrains, and sump pumps. The channel is also far enough down the basin that trickle flows, as are common in most urban basins, from various sources such lawn water, car washing, etc., may become prevalent. As noted above, trickle flows were observed in this channel during the non-growing season when the irrigation ditches were dry.

Reconstruction will require lowering the channel bottom beginning at the drop structure just upstream of Highway 44. It is expected the top of both of the existing drop structures will have to be lowered 2.5 to 3.5 feet because the existing channel depth for a distance upstream of the drops was created with a 2' high berm. Lowering the channel will allow the berm to be removed and allow positive drainage to the channel.

In additional to lowering, the drop structures will also have to be widened to fit the new channel shape. It is also estimated that two new drop structures will be required to obtain the recommended channel slope. One will be between the existing drops. The second will be a short distance downstream of the Murphy Irrigation Ditch

The proposed channel depth of 5.5' provides about 1.4' of freeboard for the 100 year design flows (1' + velocity head+-).

The existing easement is 60' feet in width. At the proposed total depth of 5.5' the proposed composite channel top width is 84' which requires additional easement.

An alternate overbank shape with 8' width and 3:1 side slopes would theoretically fit within the existing easement; however, flow depth increases to 4.4'.

The composite channel will simplify the maintenance requirements of the channel. The low flow wetland channel will not need regular mowing, rather the vegetation can remain as a low flow roughness n value of 0.065. The overbanks will need to be maintained to the characteristics of a grass lined channel.

The wetland vegetated low flow channel is conducive to water quality enhancement. The proposed low flow channel, based on shallow flow n value of 0.065, has capacity for approximately 100 cfs which is all of the 2 year storm. The 2 year storm velocity in the low flow channel is about 2.5 fps. Additional discussion related to water quality treatment in improved channels is found in Section 12.

A maintenance/access "road" should be provided along the channel per the requirements of the RCIDCM. This will require additional easement width.

If future development will provide a wider drainage route an option to be considered would be to modify the existing low flow channel without lowering the bottom, provide a wider overbank area to lessen required flow depth, and grade the adjacent property to be above the channel as necessary. Detailed survey is needed to determine if this option is feasible. This option still follows recommendation for a composite channel shape.

As noted the existing channel does not have capacity for the design flows. Normal depth for the existing channel, based on the original design information, a maintained wetland n value of 0.044, and assuming all flow is contained to the trapezoid section is 5.3'. There are no homes in the immediate vicinity of the channel; nevertheless, it is recommended the channel be maintained to keep the vegetation as short as possible for reduced flow depth prior to the channel being improved.

(End of Element 19 Narrative)

## ELEMENT 20

**Description:**

Element 20 is the double 14’ x 4’ Concrete Box Culvert under Highway 44. The box culvert was constructed by SDDOT circa 2009.

**Modeling Information:**

The following data was utilized to model the element in HMS.

28’ Bottom Rectangular Channel
n = 0.013
Length = 168’
Slope = 0.002 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 20- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 94 |
| 10 Year | 448 |
| 100 Year | 990 |
| 100 Year Velocity | 8.9 fps |

**Recommendations:**

Recommended design flow at Element 20 is the flow calculated at Junction 211. The design flows are tabulated below.

| | Element 20 Design Flows (Junction 211 Flows) (cfs) |
|---|---|
| 2 Year | 94 |
| 10 Year | 449 |
| 100 Year | 991 |
| 100 Year Velocity | 8.9 fps |

It is noted that about 10% of the Sub-basin 11 flows would not actually reach Element 20. Some of this flow would be on the south side of the highway or otherwise flows under Highway 44 through small culverts east of the box. However; for the purposes of Element 20, it has been assumed all of Sub-basin 11 contributes flow to the channel. This assumption is well within the accuracy of the modeling and calculation. It is also possible in the future that some of the Sub-basin 11 flows on the north side of the highway will be redirected to the channel rather than the small secondary culverts under Highway 44.

Minor grading improvements are recommended.

The channel berm (ditch block) on the east side of the channel at the upstream face of the Highway 44 box culverts needs to be raised about 12” to provide the required headwater for the Highway 44 box culvert (Element 20). The existing top of berm is at about 3375.5 and should be raised to about 3376.5.

The riprap stilling basin should be reconstructed with appropriate size rock and modified to be as large as reasonable to slow flows and to provide an additional settling area for sediments. Larger riprap is recommended because the existing rock has failed.

As calculated by HY8 using original design SDDOT data the box culvert has capacity to convey the 100 year design flow of 991 cfs without overtopping of Highway 44 or the raised ditch block. About 15” of freeboard is available below the roadway centerline. No freeboard is available for the ditch block. If flows overtop the ditch block they will travel east in the roadway ditch to reach twin 24” culverts under Highway 44 about 900 feet east of the box culvert. Those twin 24” pipes are located at the southeast corner of Sub-basin 11 and have capacity for about 50 cfs total.

(End of Element 20 Narrative)

## ELEMENT 21

**Description:**

Element 21 is an existing graded channel between Elements 20 and 22.

The channel is downstream of the Highway 44 box culvert and runs parallel to the south side of Highway 44.

The channel was constructed by SDDOT circa 2008.

A trickle/base flow was observed in the channel throughout the growing season. A base flow was also observed at certain times outside of the growing season. Same as with upstream channels, the majority of the trickle/base flow appears to be from "leaking" structures on the Hawthorne Irrigation Ditch. However, it is believed that high groundwater, utility underdrains, and sump pumps may be contributing to the trickle/base flow as evidenced by the presence of these flows outside of the irrigation season.

**Modeling Information:**

The following data was utilized to model the element in HMS. This channel shape was used as a typical approximation of the existing channel.

8 Point Composite Channel
Low Flow Channel: 20' Wide Bottom, 3' Deep, 6:1 Slopes
Low Flow Channel n = 0.046
Overbanks: 25' Bottom Each Side of Low Flow Channel, 2' Deep, 6:1 Slopes
Overbank n = 0.035
Length = 1330'
Slope = 0.0083 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts the wetland low flow channel down to 0.046 to account for the overall depth of flow in the entire channel. The overbank n value is for grass

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 21 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 101 |
| 10 Year | 484 |
| 100 Year | 1,056 |
| 100 Year Velocity | 5.2 fps |
| 100 Year Depth | 3.8 ft |

**Recommendations:**

The recommended design flows are the same as the routed flows given above.

Recommendations for water quality enhancement are recommended.

Channel capacity is adequate based on normal depth calculations of the design flows and the assumed n values being representative of wetland vegetation in the low flow channel and grass in the overbanks.

Total available channel depth is estimated at about 5'. Required freeboard is about 1.4' (1' plus velocity head.) Only about 1.2' of freeboard is available; however, this was judged acceptable due to the location of the channel and because no structures are nearby. If necessary, additional freeboard can be provided by raising the historic railroad embankment on the south side of the ditch.

The low flow channel has capacity for about 462 cfs based on the wetland n value of 0.065. This is far in excess of the 2 year flow. Velocity for 462 cfs is about 4.1 fps.

As noted the low flow channel has capacity far in excess of the 2 year storm. Therefore channel enhancements for water quality are recommended along and in the near vicinity of this channel. It is recommended a series of varying size micropools be excavated along the low flow portion of the channel for sediment deposition, dilution, and to provide varying types of vegetation and habitat. Small rock check dams could be installed along with the micropools to enhance filtration/sedimentation.

It is also recommended an enlarged marsh/wetland area for water quality enhancement be created in the Highway 44 right of way on the east side of the channel where it turns south to become Element 22.

(End of Element 21 Narrative)

## ELEMENT 22

**Description:**

Element 22 is an existing graded channel between Element 21 and Rapid Creek.

The channel drains in a southerly direction. Green Valley Drive crosses this channel. The Green Valley Drive is a double 12' x 6' box culvert.

A 1' high gabion drop structure is located near the box culvert inlet. A 4' high gabion drop structure is located at the upstream end of the channel just south of the historic railroad embankment.

The Little Giant Irrigation Ditch crosses the channel. An inverted siphon carries the ditch under the channel.

The channel, siphon, and box culvert were constructed by Pennington County circa 2000.

A trickle/base flow was observed in the channel throughout the growing season. A base flow was also observed at certain times outside of the growing season. Same as with upstream channels, the majority of the trickle/base flow appears to be from "leaking" structures on the Hawthorne Irrigation Ditch. However, it is believed that high groundwater, utility underdrains, and sump pumps may be contributing to the trickle/base flow as evidenced by the presence of these flows outside of the irrigation season.

Original design drawings indicate the channel as a 10' bottom, 4:1 side slopes and longitudinal grade of 0.70%. Minimum depth of the designed channel was 5'. Edge berms were used to create the design depth along part of the channel. Downstream of Green Valley Drive the channel has a minimum depth of about 7' without berms.

The downstream 850' ± of Element 22 is in the Rapid Creek 100 Year floodplain. The downstream most 550'± and the Green Valley Drive box culvert are within the floodway portion of the floodplain.

**Modeling Information:**

The following data was utilized to model the improved element in HMS.

8 Point Composite Channel
Low Flow Channel: 10' Wide Bottom , 2.5' Deep, 2:1 Slopes
Low Flow Channel n = 0.045
Overbanks: 25' Bottom Each Side of Low Flow Channel, 3.5' Deep, 4:1 Slopes
Overbank n = 0.035
Length = 2200'
Slope = 0.0070 ft/ft

The low flow n value was determined by computer model UD-CHANNELS which converts a typical low flow channel n of 0.065 down to 0.042 to account for the overall depth of flow in the entire channel. The overbank n value is for grass.

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 22 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 99 |
| 10 Year | 481 |
| 100 Year | 1,048 |
| 100 Year Velocity | 5.7 fps |
| 100 Year Depth | 4.4 ft |

**Recommendations:**

Recommended design flows at the upstream end of Element 22 are the same at the routed flows above.

Recommended design flows at the downstream end of the Element are as calculated at Junction 213. Downstream Design flows for Element 22 are tabulated below.

| | Element 22 Downstream Design Flow (cfs) (Junction 213) |
|---|---|
| 2 Year | 111 |
| 10 Year | 556 |
| 100 Year | 1231 |
| 100 Year Velocity | 6.1 fps |
| 100 Year Depth | 4.7 ft |

It is recommended the channel be improved with a composite shape for capacity and water quality purposes. The composite shape should be like the HMS modeling data or some other satisfactory configuration.

The existing channel does not have capacity to carry the design flow even if the n value is reduced to that of grass lined channel. It is assumed that future development will desire a confined channel area rather than allowing broad flooding that will occur under existing conditions. Therefore an improved channel is recommended from the upstream end of the Element to the Green Valley Drive box culvert.

It is believed the low flow portion of the composite channel will take on characteristics of a wetland channel even if the leaking irrigation structures are repaired. It is possible trickle flows may still enter the channel from high groundwater, underdrains, and sump pumps. The channel is also far enough down the basin that trickle flows, as are common in most urban basins, from various sources such lawn water, car washing, etc., may become prevalent.

Reconstruction will require lowering the channel bottom by 1’ in order to provide a minimum channel depth of 6’. Lowering the channel is made possible by removal of drop structure at the upstream face of the Green Valley Drive box culvert.

The proposed channel depth of 6’ provides about 1.3’ of freeboard for the 100 year design flows (1’ + approximate velocity head+-).

A new 1’ drop structure will be necessary at the downstream side of the Little Giant Irrigation Ditch siphon. Extra channel depth appears available upstream of the Irrigation crossing although some berming for freeboard may be necessary.

The design drawings indicate the siphon has about 65 feet of “flat” pipe under the existing 10’ wide channel. It will be necessary to warp the proposed composite section to a section that fits the over the siphon.

The existing easement is 100’ feet in width. At total depth of 6’ the proposed composite channel top width is 98’ which theoretically fits in the easement. Grading outside of the easement may be necessary for blending of the slopes.

The composite channel will simplify the maintenance requirements of the channel. The wetland low flow channel will not need regular mowing, rather the vegetation can remain as a low flow roughness n value of 0.065. The overbanks will need to maintained to grass channel characteristics.

The wetland vegetated low flow channel is conducive to water quality enhancement. The low flow channel, based on shallow flow n value of 0.065, has capacity for 111 cfs or approximately the entire 2 year storm. The 2 year flow velocity is about 3.1 fps. Additional discussion related to water quality treatment in improved channels is found in Section 12.

A maintenance/access “road” should be included along the reconstructed channel per the requirements of the RCIDCM. This will require additional easement width.

If future development will provide a wider drainage easement an option to be considered would be to use the existing low flow channel without lowering the bottom and provide a wider overbank area beginning an appropriate distance up the existing channel slopes to lessen required flow depth. This option still follows recommendation for a composite channel shape.

The existing channel slope flattens to about 0.2% downstream of the Green Valley Drive box culvert. Available existing channel depth is about 7’ downstream of the box culvert. Normal depth in the existing typical channel, assuming an n value of 0.050 and discharge of 1,230 cfs, is 7.1’. Flow depth is slighter deeper than the channel but is judged acceptable because this area is in the Rapid Creek floodway. If it is necessary at some future date to reduce flow depth it would be possible to reconfigure the channel as a composite channel by grading a 6 to 8’ wide overbank on each of the existing 6:1 side slopes and steepening the side slopes as necessary.

The 100 year design flow will overtop Green Valley Drive. The box culvert has capacity for about 1,050 cfs before overtopping. The 100 year event will have 1,095 cfs through the box and overtopping of 135 cfs at an estimated depth of about 7”.

As noted the existing channel upstream of Green Valley Drive does not have capacity for the design flows. Normal depth for the existing channel, based on the original design information, a maintained wetland n value of 0.044, and assuming the flow is confined to the trapezoid channel is 5.8’. A review of the design drawings indicates only 5’ of depth is available along part of the channel.

(End of Element 22 Narrative)

## ELEMENT 23

**Description:**

Element 23 is a 54” RCP storm sewer along Reservoir Road. This pipe drains Sub-Basin 9 into the Element 17 channel. The storm sewer was constructed in 2010 as part of the Reservoir Road reconstruction project.

**Modeling Information:**

The following data was utilized to model the existing element in HMS.

54” Pipe
n = 0.013
Length = 1500’
Slope = 0.006 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 23- HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 19 |
| 10 Year | 78 |
| 100 Year | 160 |
| 100 Year Velocity | 9.6 fps |

**Recommendations:**

Recommended design flows are the same as the routed flows given above.

No improvements to Element 23 are required for storm water flows. Improvements are recommended for the Hawthorne Irrigation Ditch Overflow as discussed in the Irrigation Ditch Section of this report.

Full flow capacity of the pipe is about 152 cfs so there will be minor flows on the street in the 100 year storm. The 100 year flow spread on each side of the street will be about 13’ which meets criteria.

(End of Element 23 Narrative)

## ELEMENT 50

**Description:**

Element 50 is an existing 48" pipe under Longview Road. The pipe begins at the Detention Pond 102 riser structure and discharges into Detention Pond 105.

Flows from a portion of Sub-basin 15 also enter this pipe.

**Modeling Information:**

The following data was utilized to model the element in HMS.

48" RCP
n = 0.013
Length = 150'
Slope = 0.0066 ft/ft

Routed flows in this element are tabulated below. These are the routed flow from Detention Pond 102 as calculated by HMS assuming full implementation of all recommendations in this report.

| | Element 50 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 0.3 |
| 10 Year | 1.1 |
| 100 Year | 26 |
| 100 Year Velocity | 7.5 fps |

**Recommendations:**

Recommended design flows at the downstream end of Element 50 are the flows calculated at Junction 250 to account for flows from Sub-basin 15. It is noted a small portion of Sub-basin 15 does not actually drain to the pipe; nevertheless, the Junction 250 flows can be used for design. The recommended design flows are tabulated below.

| | Element 50 Design Flow At Downstream End (Junction 250) (cfs) |
|---|---|
| 2 Year | 12 |
| 10 Year | 46 |
| 100 Year | 95 |
| 100 Year Velocity | 10.4 fps |
| 100 Year Depth | 2.7 |

No improvements to Element 50 are required. The pipe has capacity to carry the entire 95 cfs flow.

For the purposes of this report the Longview Road storm sewer connecting to Element 50 from the west is considered a minor system. A portion of Sub-basin 15 drains into this pipe. Because the pipe is considered a minor system it was beyond the scope of work to prepare a detailed analysis of that storm sewer and inlet system. However a cursory analysis was made.

The Longview Road storm sewer varies in size from 24” to 36”. It appears the inlets available on Longview Road and Obrien Street are inadequate to fill the pipe. However, the cursory analysis indicates the Longview Road street capacity plus the estimated flow entering the storm sewer will meet RCIDCM criteria for 10 year and 100 year spread and depth. Nevertheless, consideration should be given to adding inlets if or when future improvements are made to either Obrien Street or Longview Road Some shallow overtopping may occur at the Longview Road sag point at Element 50. The Longview Road storm sewer was constructed as part of the Reservoir Road project in 2010.

(End of Element 50 Narrative)

## ELEMENT 51

**Description:**

Element 51 is an existing 10" steel pipe with unconfined overflow in the storage yard between Detention Pond 105 and the Murphy Irrigation Ditch.

The existing pipe has capacity for only minor flows before broad sheet flooding will occur in the storage yard. The pipe and sheet flooding flows into the Murphy Irrigation Ditch creating potential downstream problems along the ditch.

**Modeling Information:**

The following data was utilized to model the element in HMS.

30" Diameter RCP
n = 0.013
Length = 200'
Slope = 0.015 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element from Detention Pond 105 as calculated by HMS assuming full implementation of all Amendment recommendations in this report.

| | Element 51 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 0.3 |
| 10 Year | 9 |
| 100 Year | 36 |
| 100 Year Velocity | 11.1 fps |

**Recommendations:**

A new storm sewer is recommended. The recommended pipe is a 30" RCP with capacity of the 50 cfs as noted above. The upstream end of the pipe is near the toe of the slope where the Detention Pond 105 outlet channel currently ends. It is recommended the pipe then be placed in a southwest direction perpendicular to the irrigation ditch. The pipe is to bridge over the irrigation ditch rather than discharging into the ditch.

The recommended design flow for the new pipe is the same as the routed flow given above. However, it is recommended the pipe be sized for about 50 cfs to provide some level of safety factor and to allow for a minor amount of additional inflow from the storage yard area.

Carrying flows over the irrigation ditch is recommended to eliminate problems that can result from the use of irrigation ditches for storm water flows. However, if permission can be obtained from the Ditch Company, it is recommended that at least the 2 year flow of 0.3 cfs be allowed to enter the ditch for the purposes of stormwater quality. This low flow would be treated by dilution and by the water being used for crop irrigation (land application).

(End of Element 51 Narrative)

## ELEMENT 52

**Description:**

Element 52 consists of a proposed grass lined channel to convey flows south from Element 51 to the ditch on the north side of Highway 44 and then continues as the existing Highway 44 ditch east to Reservoir Road.

Under current conditions Element 52 is the Murphy Irrigation Ditch and the ditch on the west side of Reservoir Road.

**Modeling Information:**

The following data was utilized to model the element in HMS.

12' Bottom Trapezoid Channel
4:1 Side Slopes
n = 0.035
Length = 1900'
Slope = 0.004 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element from Detention Pond 105 as calculated by HMS assuming full implementation of all Amendment recommendations in this report.

| | Element 52 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 0.3 |
| 10 Year | 9 |
| 100 Year | 36 |
| 100 Year Velocity | 2.3 fps |
| 100 Year Depth | 1.0 ft |

**Recommendations:**

Recommended design flows at select locations are given below. Design flows at the driveway location were determined using linear interpolation between Element 52 and Junction 251 flows. Design flows at the downstream end of Element 52 are the flows calculated at Junction 251.

| | Element 52 Recommended Design Flow For New Channel (cfs) | Element 52 Interpolated Design Flow At Driveway Crossing (cfs) | Element 52 Downstream Design Flow At Reservoir Road (Junction 251) (cfs) |
|---|---|---|---|
| 2 Year. | 1 | 15 | 40 |
| 10 Year. | 15 | 50 | 121 |
| 100 Year. | 50 | 100 | 212 |
| 100 Year Velocity | 2.5 fps | 3.1 fps | 3.8 fps |
| 100 Year Depth | 1.2 ft | 1.7 ft | 2.5 ft |

Recommended improvements consist of construction of the new ditch segment between Highway 44 and Element 51. The Highway 44 ditch section does not need to be improved. Culverts under the existing driveway to Highway 44 about 1200' west of Reservoir Road also require improvements.

As noted earlier, the channel upstream of Highway 44 to the Element 51 outlet location is a proposed new channel. Based on review of adjacent contours it is assumed this channel will not receive significant amounts of flow from the adjoining land. As such the design flow is then close to the routed flow but should be increased at least to match the recommended 50 cfs design capacity of upstream Element 51.

The new channel section is proposed to match the same section as the modeled channel but other shapes are possible. If the Element 51 trickle and low flows are not allowed to enter the Murphy Irrigation Ditch the channel should be designed in anticipation of a wetland bottom.

The existing Highway 44 ditch is adequate based on a review of the GIS aerial contours. It appears that a minimum of about 4 feet of depth is available before flow onto the roadway would occur. The private property side of the ditch is lower and may not contain the flow to the right of way. Easements may be necessary at time of platting or the property could be filled as it is developed.

The existing driveway that is located about 1200' west of Reservoir Road has a single 24" RCP culvert. It is recommended the 24" RCP be replaced with an 8' x 2' concrete box culvert or other equivalent system for the 100 year flow of 100 cfs. The 100 year storm was selected for the culvert sizing because (1) the driveway was assumed to be the single point of access when the property is developed and (2) because of terrain overtopping flows may flow onto the private property rather than overtopping into the ditch.

As described above Element 52 routes the flows in an easterly direction along Highway 44. Previous reports had shown a proposed routing to convey the flow south of Highway 44 through an area where no channel currently exists. That routing was not used because the current ditch and pipe systems drain east. This routing is more economical than creating a new channel draining south of Highway 44. Furthermore it was assumed the property owners would object to any new channel if it is not needed for their development.

(End of Element 52 Narrative)

## ELEMENT 53

**Description:**

Element 53 is the existing roadway ditch on the south side of Highway 44. Reservoir Road crosses Element 53. Detention Pond 107 discharges to Element 53.

**Modeling Information:**

The following data was utilized to model the element in HMS.

20’ Bottom Trapezoid Channel
6:1 Side Slopes
n = 0.035
Length = 1600’
Slope = 0.005 ft/ft

Routed flows in this element are tabulated below. These are the routed flows in the element from Detention Pond 107 as calculated by HMS assuming full implementation of all Amendment recommendations in this report.

| | Element 53 - HMS Routed Flow (cfs) |
|---|---|
| 2 Year | 8 |
| 10 Year | 44 |
| 100 Year | 80 |
| 100 Year Velocity | 2.7 fps |
| 100 Year Depth | 1.1 ft |

**Recommendations:**

Only minor additional flows will enter Element 53 from the adjacent sub-basin areas. As such the recommended design flows for Element 53 are the same as the routed flows given above.

Improvements to the Reservoir Road crossing are recommended. The existing culvert under Reservoir Road is a 30” RCP. Flows that exceed the capacity of the existing pipe will split away from drainage system and flow south along Reservoir Road. It is recommended an additional 30” RCP be installed to bring total capacity of the crossing to about 80 cfs.

The channel is adequate with no improvements necessary. If the Element 51 trickle and low flows are not allowed to enter the Murphy Irrigation Ditch it should be anticipated that Element 53 will likely take on characteristics of a wetland channel.

(End of Element 53 Narrative)

# 8. DETENTION PONDS

## 8.1 INTRODUCTION AND MODELING DATA

This section of the report presents discussion for the each of the Detention Ponds in the DBDPA.

Appendix A contains the detention pond modeling input data for Existing Condition ponds. Appendix B contains the detention pond modeling input data for the recommended DBDPA conditions.

Input data for detention ponds in this study is table of storage versus discharge for each pond.

Storage data for the pond was determined by digitizing PDF files of as-built or original drawings were possible. Aerial contours were used for storage data at Detention Ponds 102 and 105. Storage data for new ponds was estimated from aerial contours using engineering judgment for grading.

Discharge curves for pipes were developed using HY8 culvert analysis software. Discharge data for orifices and weirs used standard engineering equations. Discharge curves assume unobstructed flow conditions.

Certain culverts and pipes under roadways were not modeled as detention ponds or backwater elements. Rather they were assumed simply to be a portion of the adjacent routing elements. Modeling limitations, insignificant backwater or storage, minor flow lengths, and/or overtopping characteristics warrant this assumption.

## 8.2 DETENTION POND OVERVIEW

Unless otherwise noted in the following narratives a minimum of 1 foot of freeboard has been provided below the rim of all riser structures. This minimum freeboard is necessary because orifice controlled flows are important in the overall plan. Overtopping flow into risers caused by orifice clogging, modeling uncertainties/assumptions, modeling limitations especially as related to ponds that are in series, development uncertainties, or higher than expected flows, could potentially have serious effects on downstream elements and ponds.

As noted earlier, capacity problems downstream of Twilight Drive became apparent during the study. During the analysis it was determined that reductions in flows from ponds upstream of Twilight Drive was of significant benefit in reducing flows in Elements 14 and 15 to manageable rates, even though improvements to those elements would still be necessary. As such the recommendations for ponds upstream of Twilight Drive not only correct deficiencies in existing ponds but also reduce flows to Elements 14 and 15 to nearly the maximum extent possible.

**As noted earlier in Section 3.8 there are a number of minor or small onsite detention ponds located in the study area. Additional small onsite ponds are expected to be constructed in future developments as required to meet the RCIDCM requirement of maintaining runoff to existing conditions prior to implementation of all improvements in this DBDPA. The added small ponds are also anticipated necessary as part of the requirements for Water Quality Capture Volume for storm water treatment.**

All existing and future minor, small, and onsite ponds have been ignored in this study because (1) the ponds are too small to be accurately included in the overall analysis, (2) history shows some small ponds may become abandoned, (3) private ponds will likely not be maintained to design conditions, and (4) uncertainty as to where future small ponds would be located.

Unless otherwise noted in detailed discussions the only detention ponds included in the HMS analysis are the regional ponds described in this report section.

Detention Ponds included in the DBDPA are considered regional type ponds. These are existing Detention Ponds 100, 101, 102, 103, 104, and 105 plus recommended Detention Ponds 106 and 107.

## 8.3 SUMMARY RESULTS

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for existing land use conditions existing hydraulic conditions are given on Table 5 on Page 30 at the rear of Section 6.

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for future land use conditions and the proposed future DBDPA hydraulic conditions are given on Table 6 on Page 31 at the rear of Section 6. These values are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

Summarized results that are direct printouts from the HMS run are also included in Appendix A for existing land use and existing hydraulic conditions.

Summarized results that are direct printouts from the HMS run are also included in Appendix B for future land use and future hydraulic conditions. These printouts are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

A summary of the recommended improvements is found in Section 13 of this report and is entitled Major Recommendations Summary, Cost Estimate, and Prioritization

Figures 16, 17, and 18 are enlarged site plan drawings illustrating the areas where major recommendations are proposed.

Hydrographs of the DBDPA condition flow elements, detention ponds, and junctions are included in Appendix C.

## 8.4 DETAILED DETENTION POND DISCUSSION

Detailed discussion for each of the Detention Pond Elements modeled in the study follows. Each element includes:

- Description of the Element,
- Modeling Information used in the HMS model, and
- Discussion and Recommendations

Refer to Section 7 for Channel and Pipe Elements, Section 9 for Junctions, and Section 10 for modeled "minor" basins.

## DETENTION POND 100

**Description:**

Detention Pond 100 is an existing detention cell located north of Avenue A and east of Degeest Drive. Existing storage capacity in this pond nearly meets the requirements of the State of South Dakota Small Dams Regulations. Available storage at top of dam elevation is 48.9 acre feet whereas the State of South Dakota begins regulation if capacity is 50 acre-feet.

The existing outlet consists of a 48" RCP riser with an 18" low flow pipe connected to the riser. A 36" RCP serves as the outlet from the riser system. A 60' spillway is available for overflow.

The following issues became apparent during review and analysis of Detention Pond 100.

- The existing pond is very close to being regulated as a dam by the State of South Dakota because of storage capacity.
- Modeling of the basin indicated that upon full upstream development the pond has only about 6" of freeboard below the spillway. This is consistent with freeboard information shown on the original design drawings. This was judged to be inadequate freeboard given the large storage capacity of this dam and the potential for serious downstream issues should the pond overtop.
- Water Quality design was not incorporated into the original design as that was not a requirement of the City at the time the pond was constructed.
- The trash rack on the existing low flow inlet pipe is in a state of disrepair and is susceptible to plugging.
- A trash rack is present at the outlet end of the discharge pipe. This rack could create plugging problems if materials would enter the upstream end of the outlet pipe.
- Trees are growing very close to the existing outlet pipe and riser.
- Wetland vegetation is becoming well established in the majority of the pond bottom
- The lot encompassing this pond is defined as a drainage easement. The lot was previously under the ownership of Pennington County which would have allowed for multiple use in the area. The lot is now shown as being privately owned.
- Further reduction and delay of peak flows from Pond 100 were of significant benefit to restricted flow capacity areas downstream of Twilight Drive.

**Modeling Information:**

The following data was utilized to model Detention Pond 100 in HMS. The data is based on the recommended improvements.

| DETENTION POND 100 MODELING DATA (INCLUDES REGRADING OF POND BOTTOM AND MODIFIED RISER) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre-Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3209 | 0.0 | 0 | 6" Orifice FL 3209 |
| 3213 | 2.7 | 1.8 | 12" Orifice FL 3213 |
| 3214 | 5.0 | 4 | |
| 3215 | 7.9 | 7 | |
| 3216 | 11.0 | 8.5 | 24" Orifice FL 3416 |
| 3217 | 14.7 | 18 | 24" Orifice FL 3417 |
| 3218 | 19.0 | 34 | |
| 3219 | 23.5 | 48 | |
| 3220 | 28.2 | 60 | Top of 48" Riser EL 3220 |
| 3221 | 33.0 | 104 | |
| 3222 | 38.1 | 108 | |
| 3222.5 | 40.7 | 110 | 60' Spillway, EL 3222.5 |
| 3223 | 43.4 | 167 | |
| 3224 | 48.9 | 451 | Top of Dam 3224 |

Based on the Detention Pond 100 Modeling Data above the resulting water elevations, storage, and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 100 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 45 | 2 | 3213.1 | 2.9 |
| 10 Year | 146 | 8 | 3215.8 | 10.5 |
| 100 Year | 282 | 42 | 3218.6 | 21.7 |

**Recommendations:**

As noted above there are several issues related to this pond and improvements are recommended.

It is recommended the pond bottom be regraded to provide a defined low flow channel and to provide for Extended Detention Water Quality Volume. Four feet of storage depth with 2.7 acre-feet of storage has been included in the modeling data for the Extended Detention. The proposed stage discharge curve assumes a 6" orifice for "dummy" modeling of extended detention discharge. Further discussion of Extended Detention is found in the Water Quality section of this report.

Pond regrading is recommended for the Extended Detention volume in order to reduce the area of the pond bottom that will be inundated on this frequent basis. The excavated material will be placed elsewhere in the bottom so the total storage volume remains essentially the same as existing.

Bottom regrading should include a defined low flow channel. The low flow channel should have a flat grade to promote wetland type characteristics for water quality enhancement. Regrading should include removal of trees that are growing near the riser structure.

The bottom of the Extended Detention volume and thus the 6” low flow orifice are proposed as being 1’ lower than the existing outlet elevation of 3210. This is possible because the existing low flow pipe slopes sharply down to the bottom of the riser structure.

Modifications to the existing riser are also recommended. The existing 18” low flow pipe will be lowered and modified to act as the 6” orifice for the Extended Detention. Additional round orifices should be installed in riser of the size and elevation as noted on the outlet table above. The top of the riser elevation is not being adjusted.

Appropriately sized debris racks are recommended for the orifices and the riser. Removal of the trash rack on the outlet end is recommended. If a trash rack on the outlet end is mandated it should be hinged to allow the rack to freely swing open as a safety factor against clogging.

The 100 year water elevation provides about 1.4 foot of freeboard below the riser top and about 4 feet of freeboard below the spillway elevation.

It is also recommended the current property owner be approached about the property being returned to public ownership to provide opportunities for open space or multiple used type recreation.

(End of Detention Pond 100 Narrative)

## DETENTION POND 101

**Description:**

Detention Pond 101 is an existing detention cell located east of Ziebach Street and south of Homestead Street. This pond was constructed as part of the adjacent subdivision projects.

The existing outlet consists of a 36" RCP culvert and a 20' wide spillway. The spillway is lined with cable concrete.

The following issues became apparent during review and analysis of Detention Pond 101.

- The City of Rapid City noted the original design and construction of this pond required a certain level of approval by the USCOE due to wetlands. The City noted this is why the pond spillway is not in the location indicated on the original subdivision drawings. Because of the USCOE issue, the City stated that any improvements to this pond were not to include any grading of the pond bottom, additional pipes, or relocation of the pipes.
- The existing spillway armoring has a flat cross section and will not confine overtopping flows to the armored section.
- The original design report for the pond indicates the spillway is utilized for the 100 year design event, rather than being only an emergency overflow. This was verified by the HMS model in this study.
- Wetland vegetation is starting to emerge in the bottom of the pond. The City noted that local residents were aware this pond may become a wetland bottom when it was constructed.
- Water Quality design was not incorporated into the original design as that was not a requirement of the City at the time the pond was constructed.
- The trash rack on the existing outlet pipe is in a state of disrepair and was nearly completely covered with debris.
- Access for maintenance equipment to the dam embankment and outlet pipe is difficult.
- The pond is within a lot that is dedicated as a major drainage easement. The lot is currently under private ownership.

**Modeling Information:**

The following data was utilized to model Detention Pond 101 in HMS. The data is based on the recommended improvements.

| DETENTION POND 101 MODELING DATA (INCLUDES IMPROVED OUTLET PIPE AND SPILLWAY RAISE) | | | |
|---|---|---|---|
| ELEVATION (feet) | STORAGE (Acre-Feet | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3221.7 | 0 | 0 | 12" Orifice Flow Line |
| 3222 | 0.1 | 2 | |
| 3223 | 0.48 | 4 | |
| 3224 | 1.07 | 6 | |
| 3225 | 1.79 | 7 | 12" Orifice Flow Line |
| 3226 | 2.59 | 11 | |
| 3227 | 3.45 | 14 | Top of New 72" RCP Riser |
| 3228 | 4.39 | 78 | |
| 3229 | 5.40 | 92 | |
| 3230 | 6.50 | 102 | |
| 3231 | 7.67 | 110 | Raised 20' Spillway, EL 3231 |
| 3232 | 8.93 | 182 | |
| 3232.5 | 9.60 | 230 | |
| 3233 | 10.3 | 296 | Top of Dam |

Based on the Detention Pond 101 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 101 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 28 | 6 | 3224.5 | 1.4 |
| 10 Year | 110 | 54 | 3227.6 | 4.0 |
| 100 Year | 209 | 101 | 3229.9 | 6.4 |

**Recommendations:**

It is recommended that a 72" diameter riser structure be placed at the inlet end of the 36" RCP discharge culvert. The proposed riser replaces the existing flared end. The 36" RCP connection to the riser should be a grooved end opening. The riser should have orifices as indicated on the table of modeling data. The 12" low flow orifice at elevation 3221.7 is primarily for low flow control but also creates limited Extended Detention for water quality treatment. Top of riser is set at elevation 3227.

Appropriately sized debris racks are recommended for the orifices and the riser. Removal of the trash rack at the outlet end is recommended. If a trash rack on the outlet end is mandated it should be hinged to allow the rack to freely swing open as a safety factor against clogging.

The spillway is no longer needed for design flows and will act only as an emergency overflow.
The recommendation for the spillway is to raise the elevation by 1 foot from 3230 to 3231. The cable concrete blanket should be reinstalled after the grading is accomplished for this. The cable concrete should be shaped to confine flow to the lining. The embankment outside of the spillway

should be raised from current elevation 3232.5 to at least 3233.0. A wider spillway could be considered in lieu of raising the dam.

The modeled spillway does not have capacity for the 100 year event due to site restrictions. This was judged reasonable for modeling because approximately 1.1' of freeboard exists between the calculated 100 year water elevation and spillway elevation. It is noted there are safety factors related to this spillway capacity in that the proposed top of dam is 3.1' above the 100 year water elevation and upstream Detention Ponds 101 and 103 have significant freeboard for maintaining orifice controlled flow. A higher capacity spillway should be considered at final design if detailed surveys indicate adequate room is available or if the upstream improvements which greatly reduce flows have not been implemented.

It is recommended that the pond bottom continue to be allowed to take on wetland characteristics for water quality issues. A wetland bottom will have no effect on the detention characteristics of the pond. As noted above the local residents were aware the pond may have a wetland bottom.

(End of Detention Pond 101 Narrative)

## DETENTION POND 102

**Description:**

Detention Pond 102 is an existing detention pond located north of Longview Road and east of Reservoir Road. This pond is also a lake that holds water year round. A trickle flow was observed discharging from the pond even during dry weather conditions.

The following issues became apparent during review and analysis of Detention Pond 102.

- GIS contours had to be used for stage storage calculations because as built drawings were not available.
- Many homes are located around the pond. Home elevations could only be roughly approximated from the GIS contours.
- It was not possible to determine the exact characteristics of the existing outlet system. Based on a review of a 1998 topographic map submitted as part of the Trailwood Village Subdivision project it is believed there is a 6" diameter pipe extended under water into the lake. The drawing indicates the 6" pipe is then connected to a manhole system in the dam which discharges to a 24" CMP. The drawing indicates the 6" pipe is valved.
- The outlet system manhole has an open top and is full of debris.
- There is currently no secondary pipe or overflow into the manhole structure, any excess flows would spill over the dam.
- The existing normal pool elevation is at about elevation 3113.8. The top of the existing dam embankment is at about elevation 3118.0. There is no defined spillway.
- The dam cannot be raised in elevation due to existing lakeside development.
- The dam embankment is generally devoid of vegetation because of vehicle traffic by recreation users of the lake.
- The lake and various adjacent areas are being used as a pubic fishing/recreation area even through the property is under private ownership.
- With future land use conditions and existing pond conditions, the HMS model calculated the pond as being close to overtopping during the 100 year storm.
- No easement or drainage lot exists for this detention pond.

**Modeling Information:**

The following data was utilized to model Detention Pond 102 in HMS. The data is based on the recommended improvements.

| DETENTION POND 102 MODELING DATA (INCLUDES RECOMMENDED OUTLET IMPROVEMENTS) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre-Feet | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3113.8* | 0.00 | 0.0 | 6” Orifice FL 3113.8 |
| 3114 | 0.70 | 0.1 | |
| 3116 | 8.50 | 1.3 | Top New 30” RCP Riser |
| 3117 | 13.00 | 26.0 | |
| 3118 | 17.50 | 35.0 | Top of Dam |
| 3119 | 22.5 | 762.0 | |

*Normal top of water elevation = 3113.8

Based on the Detention Pond 102 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 102 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 18 | 0.3 | 3114.3 | 1.9 |
| 10 Year | 77 | 1.1 | 3115.7 | 7.3 |
| 100 Year | 166 | 26 | 3117.1 | 13.1 |

**Recommendations:**

It is recommended the existing low flow pipe system and manhole system be completely reconstructed. The existing low flow pipe will be replaced with a new low flow pipe controlled by 6” orifice. The existing junction manhole that connects the 6” low flow pipe to the outlet pipe should be removed and replaced with a 30” RCP riser structure. The rim of the new riser is proposed at elevation 3116.0, or about 2.2 feet above normal pool elevation.

A 36” RCP should be installed between the riser standpipe and the Longview Road storm sewer located a short distance downstream. A 36” RCP was stubbed north of Longview Road for this connection as part of the recent street project.

The top of dam should also be smoothed to elevation 3118 for a length of 240’. This is the same elevation as currently exists and approximates the available distance for the embankment. The lake side of the embankment should be armored with riprap for protection against wave action. It is recommended that recreational vehicular traffic be prohibited on the embankment due to the evidence of embankment damage during wet periods.

No dedicated spillway, other than the 240’ long embankment, is proposed. This was judged acceptable due to the low height of the dam, approximate 1 foot of freeboard above the 100 year water elevation, and because any overflows on the 240’ long dam would be shallow.

It is necessary to perform a detailed topographic survey before any design work to verify stage-storage data and to determine the elevations of nearby homes and structures.

(End of Detention Pond 102 Narrative)

## DETENTION POND 103

**Description:**

Detention Pond 103 is an existing detention pond located about 1,300 feet north of Homestead Street and west of Reservoir Road.

The existing outlet consists of a 60” RCP riser with an 18” low flow pipe and 30” RCP secondary flow pipe connected to the riser. A 42” RCP serves as the outlet from the riser system. A 10’ spillway is available for overflow.

The following issues became apparent during review and analysis of Detention Pond 103.

- The 100 year storm will fill to within a few inches of the riser top assuming fully developed upstream conditions and existing pond conditions.
- The embankment has a 55’ wide top which is conducive to raising top of dam elevation.
- Reducing flow from this pond has a significant beneficial impact on downstream Detention Ponds101 and 104.
- Water Quality design was not incorporated into the design as that was not a requirement of the City at the time the pond was constructed.
- No easement or drainage lot exists for this detention pond.

**Modeling Information:**

The following data was utilized to model Detention Pond 103 in HMS. The data is based on the recommended improvements.

| DETENTION POND 103 MODELING DATA (INCLUDES REGRADING OF POND AND MODIFIED OUTLET | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3252 | 0 | 0 | 6” Orifice FL 3252 |
| 3256 | 3.9 | 1.8 | 12” Orifice FL 3256 |
| 3257 | 5.3 | 4.1 | |
| 3258 | 7.7 | 8.0 | 18” Orifice FL 3258 |
| 3260 | 12.1 | 15.0 | |
| 3262 | 17.2 | 21.0 | |
| 3263 | 20.5 | 26.0 | 18” Orifice FL 3263 |
| 3264 | 23.1 | 33.0 | |
| 3265 | 26.4 | 40.0 | |
| 3266 | 29.9 | 47.0 | |
| 3267 | 33.6 | 52.0 | Top Elev. of 60” Riser |
| 3268 | 37.6 | 106 | FL of 25’ Spillway |
| 3269 | 41.9 | 268 | |
| 3269.5 | 44.1 | 321 | Top of Dam |

Based on the Detention Pond 103 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 103 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 71 | 2 | 3256.0 | 3.7 |
| 10 Year | 298 | 16 | 3260.3 | 12.7 |
| 100 Year | 612 | 42 | 3265.3 | 27.2 |

**Recommendations:**

It is recommended the pond bottom be regraded to provide a defined low flow channel and to provide for Extended Detention Water Quality Volume. It is estimated this grading will add 0.60 acre feet of storage which is accounted for in the above storage discharge table.

Four feet of storage depth with 3.9 acre-feet of storage is included in the modeling data for the Extended Detention. The proposed stage discharge curve assumes a 6" orifice for "dummy" modeling of the extended detention discharge. Further discussion of Extended Detention is found in the Water Quality section of this report.

It is recommended the riser structure be modified to provide the orifice arrangement described in the table of modeling data. The top of the riser also will be raised 2' from 3265 to 3267. The 6" low flow orifice at elevation 3252 is intended to simulate extended detention for water quality treatment for HMS modeling only. Final extended detention orifice sizing is necessary at final design.

Appropriately sized debris racks are recommended for the orifices and the riser. Removal of the trash rack at the outlet end is recommended. If a trash rack on the outlet end is mandated it should be hinged to allow the rack to freely swing open as a safety factor against clogging.

It is expected that the low flow channel grading will result in a flat longitudinal slope which overtime may become a linear wetland water quality feature.

It is recommended that the top of embankment be raised from elevation 3268.0 to elevation 3269.5. A new 25' wide graded spillway is recommended at elevation 3268.0. The current spillway elevation is at 3266.5.

It is noted that significant freeboard, about 1.7 feet, has been provided between the 100 year water elevation and the riser top. Significant freeboard was judged necessary because any flows that would spill into the riser would seriously impact downstream Detention Ponds 101 and 104 which in turn would have downstream consequences. The freeboard provides a margin of safety against flows higher than the 100 year storm, uncertainty in final upstream development, potential orifice clogging, and modeling uncertainties.

The modeled emergency spillway does not have capacity for the 100 year flow. This was judged acceptable for modeling because approximately 3 feet of freeboard and about 10 acre feet of storage are available between the 100 year water elevation and spillway flowline. The spillway is also 2.5 times as wide as the existing spillway.

A safety factor analysis was also made whereby the pond was assumed to be full to 3265 when the 100 year event happens, essentially simulating back to back 100 year storms. That safety factor analysis indicated the second 100 year event could be safety passed through the outlet works, including spillway, without overtopping the dam.

A larger spillway should be considered at final design if adequate space is available.

It is also recommended that adequate access be provided for maintenance when adjoining property is platted.

(End of Detention Pond 103 Narrative)

## DETENTION POND 104

**Description:**

Detention Pond 104 is an existing detention pond located east of Ziebach Street and north of Homestead Street. Homestead Street creates the pond embankment. This pond was constructed as part of adjacent subdivision projects.

The existing outlet consists of a 30” RCP culvert and a 20’ wide spillway. The spillway is lined with cable concrete and discharges onto Homestead Street.

The following issues became apparent during review and analysis of Detention Pond 104.

- The existing spillway discharges onto Homestead Street. No provision was made for armoring of the roadway embankment on the downstream side of the street where the spillway flows would discharge from the street.
- The 100 year storm event will result in flows through the spillway under current pond conditions with future land development.
- Significant headcut erosion is occurring where the main channel enters at the north end of the pond due to the steep graded channel slope. This segment of channel slope is on the order of 12%.
- Wetland vegetation is starting to emerge in the bottom of the pond.
- Water Quality design was not incorporated into the design as that was not a requirement of the City at the time the pond was constructed.
- The trash rack on the existing outlet pipe is in a state of disrepair and was nearly completely covered with debris.
- The pond is within a lot that is dedicated as a major drainage easement. The lot is currently under private ownership.

Improvements to Pond 104 consisting of modifications to the outlet and minor grading are recommended.

**Modeling Information:**

The following data was utilized to model Detention Pond 104 in HMS. The data is based on the recommended improvements.

| DETENTION POND 104 MODELING DATA (INCLUDES IMPROVED OUTLET PIPE AND MINOR GRADING) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre-Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3226 | 0 | 0 | 12” Orifice FL 3226 |
| 3227 | 0.08 | 1 | |
| 3228 | 0.30 | 5 | |
| 3229 | 0.60 | 6 | |
| 3230 | 1.10 | 7 | 24” Orifice FL 3230 |
| 3231 | 1.70 | 18 | |
| 3232 | 2.30 | 29 | |
| 3233 | 3.00 | 31 | |
| 3234 | 3.80 | 37 | |
| 3235 | 4.70 | 41 | |
| 3236 | 5.70 | 46 | |
| 3237 | 6.80 | 48 | |
| 3238 | 8.00 | 53 | Top Elev. of 72” Riser |
| 3239 | 9.30 | 79 | FL of 20’ Spillway |
| 3240 | 10.70 | 158 | |
| 3240.2 | 11.00 | 189 | Top of Dam Embankment |

Based on the Detention Pond 104 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 104 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 39 | 6 | 3229.4 | 0.8 |
| 10 Year | 133 | 31 | 3232.5 | 2.8 |
| 100 Year | 247 | 47 | 3236.8 | 6.5 |

**Recommendations:**

It is recommended that the pond bottom be regraded to provide a defined low flow channel. It is estimated this grading will add about 0.5 acre feet of storage which is accounted for in the above storage discharge table. It is expected that the low flow channel grading will result in a flat longitudinal slope which overtime will become a linear wetland water quality feature.

Four feet of storage depth with 1.1 acre feet of storage, controlled by a 12” orifice, is included in the modeling data to provide low flow control and incidental Extended Detention water quality storage.

It is recommended that a 72” diameter riser structure be placed at the inlet end of the 30” RCP discharge culvert. The 30” RCP connection to the riser should be a grooved end opening. The riser should have orifices as indicated on the table of modeling data. Top of riser is set at elevation 3238.

Appropriately sized debris racks are recommended for the orifices and the riser. Removal of the trash rack at the outlet end is recommended. If a trash rack on the outlet end is mandated it should be hinged to allow the rack to freely swing open as a safety factor against clogging.

The recommendation for the spillway is to raise the elevation by 1 foot from 3238 to 3239. The cable concrete blanket should be reinstalled after the grading is accomplished for this. The embankment outside of the spillway can remain at current elevation 3240.2

The spillway does not have capacity for the 100 year event. This was judged acceptable for modeling because approximately 2.3' of freeboard exists for the 100 year water elevation, because the top of embankment is only slightly higher than the roadway, and because upstream Detention Pond 103 has significant freeboard for maintaining orifice controlled flow.

It has also been judged that armoring of the downstream roadway embankment for possible overtopping flows is not necessary because of the available freeboard.

As noted earlier the upstream channel is experiencing severe headcutting where it drops into the pond. This channel should be armored as part of the improvements to Element 2 that are expected to occur during adjacent subdivision development.

It is also recommended that adequate access be provided for maintenance when adjoining property is platted.

(End of Detention Pond 104 Narrative)

## DETENTION POND 105

**Description:**

Detention Pond 105 is an existing detention pond located south of Longview Road and west of Reservoir Road. This pond is also a lake that holds water year round. A trickle flow was observed discharging from the lake even during dry weather conditions.

The following issues became apparent during review and analysis of Detention Pond 105.

- There are no as built drawings or surveys of the pond. The existing outlet was assumed to be a 5' graded weir.
- The existing low point on the top of dam was assumed to be at elevation 3111.0 from interpolation of GIS contours.
- Stage storage data had to be determined from GIS contours.
- There is no emergency spillway.
- A drainage easement exists for this pond.

Improvements to Pond 105 are recommended. The improvements will better define the outlet characteristics and will extend resident time in pond for water quality improvements.

**Modeling Information:**

The following data was utilized to model Detention Pond 105 in HMS. The data is based on the recommended improvements.

| DETENTION POND 105 MODELING DATA (INCLUDES RECOMMENDED OUTLET IMPROVEMENTS) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre-Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3108.1 | 0.00 | 0.0 | 6" Orifice FL 3108.1 |
| 3109 | 1.8 | 0.7 | New 10' Weir F.L. 3109 |
| 3110 | 3.6 | 31 | |
| 3111 | 6.3 | 87 | |
| 3112 | 9.4 | 158 | Top Of Dam |

*Normal top of water elevation = 3108.1

Based on the Detention Pond 105 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 105 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 12 | 0.3 | 3108.5 | 0.8 |
| 10 Year | 46 | 9 | 3109.5 | 2.3 |
| 100 Year | 95 | 36 | 3110.1 | 3.8 |

**Recommendations:**

It is recommended the outlet works be modified by constructing an improved structural overflow weir. The characteristics of the weir shall be such that a 6" diameter orifice at flow line 3108.1 drains through the weir. The weir itself is recommended to be 10' long at elevation 3109.0. The orifice is intended to extend the resident time in the pond for water quality purposes.

The weir will spill into the existing channel that directs flows southwest of the dam. This channel appears to be a wetland so replacing the channel with a pipe system was not investigated.

The top of the dam should be raised to elevation 3112.0. Based on review of the GIS contours this should not be a significant undertaking as much of the embankment is already at this elevation. Raising the dam to this elevation results in about 2 feet of freeboard between the 100 year water elevation and top of dam. This also allows the outlet works to pass an emergency 100 year discharge and still maintain 1 foot of freeboard to top of dam. The additional freeboard was judged appropriate because buildings are located at the toe of the dam.

It is necessary to perform a detailed topographic survey before any design work to verify stage-storage data.

(End of Detention Pond 105 Narrative)

## DETENTION POND 106

**Description:**

Detention Pond 106 is a proposed new detention pond at the mouth of Sub-Basins 5. The pond is located about 1,000' north of Homestead Street and about 500' east of Degeest Street.

This pond has never been proposed in any previous DBDP but has been determined necessary in this DBDPA. The pond is needed to reduce peak flows that reach Detention Pond 100. This in turn allows Detention 100 to be improved as described earlier.

The following issues became apparent during review and analysis of the need for Detention Pond 106.

- Downstream Pond 100 is essentially at the criteria for State of South Dakota Small Dams and recommendations are proposed for that dam. Pond 106 is needed so Pond 100 can be modified for Extended Detention and so Pond 100 has appropriate freeboard.

**Modeling Information:**

The following data was utilized to model Detention Pond 106 in HMS. The data is based on the recommended improvements.

| DETENTION POND 106 MODELING DATA (DATA IS FOR PROPOSED POND AND OUTLET) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre- Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3253 | 0 | 0 | 6" Orifice FL 3253 |
| 3259 | 3.8 | 2.5 | 12" Orifice FL 3259 |
| 3260 | 5.2 | 4.5 | |
| 3262 | 9.0 | 9.0 | 2 - 24" Orifices FL 3262 |
| 3264 | 13.6 | 41.0 | |
| 3266 | 19.1 | 65.0 | |
| 3268 | 25.5 | 83.0 | |
| 3269 | 29.0 | 141.0 | Top Elev. of 60" Riser |
| 3270 | 32.9 | 231.0 | Top of Dam Estimate |

Based on the Detention Pond 106 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 106 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 66 | 2 | 3259.0 | 3.8 |
| 10 Year | 239 | 24 | 3263.0 | 11.2 |
| 100 Year | 472 | 71 | 3266.7 | 21.2 |

**Recommendations:**

The proposed location of Pond 106 appears to be a reasonable location for a future roadway. A roadway in this location, if determined appropriate by the developers, could be used for the dam embankment.

The recommended pond characteristics are as indicated in the table of modeling data. It is recommended that the pond bottom include a defined low flow channel and Extended Detention Water Quality Volume. Six feet of storage depth with 3.8 acre-feet of storage is included in the modeling data for the Extended Detention. The proposed stage discharge curve assumes a 6" orifice for "dummy" modeling of extended detention discharge. Further discussion of Extended Detention is found in the Water Quality section of this report.

The outlet works consist of the recommended riser tabulated above. The discharge pipe from the riser is proposed as a 48" RCP at flow line 3252 which is one foot below the 6" low flow orifice. Appropriately sized debris racks are recommended for the orifices and the riser.

The proposed top of the riser provides approximately 2 feet of freeboard above the 100 year water elevation. Significant freeboard was judged necessary because any flows that would spill into the riser could seriously impact downstream Detention Pond 100 which in turn would have downstream consequences. The freeboard provides a margin of safety against flows higher than the 100 year storm, uncertainty in final upstream development, potential orifice clogging, modeling uncertainties, and the anticipation that there will not be an emergency spillway if a road is placed on the embankment.

The modeling data does not include any spillway. A spillway or potentially more freeboard between riser top and top of dam is a function of the final design and whether or not the embankment is a roadway.

The final design of the dam must include provisions for maintenance access.

(End of Detention Pond 106 Narrative)

## DETENTION POND 107

**Description:**

Detention Pond 107 is a proposed new detention pond at the mouth of Sub-Basin 12. The pond is located at the northwest corner of the intersection of Highway 44 and Reservoir Road.

This pond has never been proposed in any previous DBDP but has been determined necessary in this DBDPA. The pond is needed to reduce peak flows that reach the Highway 44 culverts such that the culverts do not need to be upsized. Sub-basin 12 is expected to have a significant amount of commercial development and this small regional pond is judged more appropriate than various onsite ponds that would otherwise be required to meet the requirement of no increase in flows from Sub-basin 12. This pond will also serve as a regional Water Quality pond.

Detention Pond 107 does not provide a significant reduction in peak flows at downstream Junctions 212 and 213. Rather the pond is designed around the capacity of the Highway 44 culverts and to provide water quality benefits.

The following issues became apparent during review and analysis of the need for Detention Pond 106.

- The culverts crossing under Highway 44 in this area are undersized if detention is not created. The pipes have capacity for less than the 10 year storm.
- Overtopping flows would flow south over Highway 44 and east over Reservoir Road.
- Water quality improvements will be necessary when the Sub-basin 12 is developed and the proposed detention pond will provide the opportunity to use Extended Detention.

**Modeling Information:**

The following data was utilized to model Detention Pond 107 in HMS. The data is based on the recommended improvements.

| DETENTION POND 107 MODELING DATA (DATA IS FOR PROPOSED POND AND OUTLET) | | | |
|---|---|---|---|
| ELEVATION (Feet) | STORAGE (Acre- Feet) | TOTAL DISCHARGE (cfs) | OUTLET NOTES |
| 3081.5 | 0 | 0 | 6” Orifice FL 3081.5 |
| 3082 | 0.1 | 0.5 | |
| 3084 | 1.2 | 1.4 | Top New Riser for Existing 30” RCP at intersection |
| 3085.2 | 2.5 | 38 | FL for existing 30” RCP located about 480’ west |
| 3086 | 3.5 | 47 | |
| 3088 | 6.0 | 83 | |
| 3090 | 8.7 | 106 | Top of Dam |

Based on the Detention Pond 107 Modeling Data above the resulting water elevations, storage and discharge are as follows. These results are based on full implementation of all Amendment recommendations in this report.

| DETENTION POND 107 DBDPA MODELING RESULTS | | | | |
|---|---|---|---|---|
| EVENT | PEAK INFLOW (cfs) | PEAK OUTFLOW (cfs) | WATER ELEVATION (feet) | STORAGE (acre-feet) |
| 2 Year | 40 | 8 | 3084.2 | 1.4 |
| 10 Year | 121 | 44 | 3085.8 | 3.1 |
| 100 Year | 212 | 80 | 3087.8 | 5.8 |

**Recommendations:**

The recommended pond characteristics are as indicated in the table of modeling data. It is recommended that the pond bottom include a defined low flow channel and Extended Detention Water Quality Volume. Extended storage depth of 2.5' with 1.2 acre-feet of storage is included in the modeling data for the Extended Detention. The proposed stage discharge curve assumes a 6" orifice for "dummy" modeling of extended detention discharge. Further discussion of Extended Detention is found in the Water Quality section of this report.

Two existing 30" RCP culverts under Highway 44 serve as the outlet. One pipe is at the intersection of Highway 44 and Reservoir Road. It is intended that this pipe will be modified with a riser structure at the inlet end with the 6" orifice at flow line 3081.5. Top of the riser will be at elevation 3084.0 which is the top of the proposed Extended Detention storage pool. Appropriately sized debris racks are recommended for the orifices and the riser.

The second 30" culvert is located about 480 feet west of the intersection. No improvements are proposed for this pipe.

The dam is created by the Highway 44 embankment. The centerline of Highway 44 is at about elevation 3090.0 at this location. Top edge of pavement is at about 3088.5 which is above the 100 year water elevation.

A short embankment is needed along the east side of the pond because Reservoir Road is lower than Highway 44.

The modeling data does not include any spillway because the overtopping flows would be over the highway.

Elevation data for this pond was determined from SDDOT plans. Elevations were adjusted down by 1.6' from the SDDOT 1988 datum to 1929 datum used in this report.

(End of Detention Pond 107 Narrative)

# 9. JUNCTION ELEMENTS

## 9.1 INTRODUCTION AND MODELING DATA

Junctions are not hydraulic routing elements, rather they serve to summarize flows. In previous studies using CUHP/UDSWM the Junctions were known as Direct Flow Elements. To remain consistent with HMS literature the Junction terminology is used in this report.

No data input is necessary for Junctions.

Junctions are elements with one or more inflows and only one outflow. All inflow is added together to produce the outflow by assuming zero storage at the junction. There are no time lags between the inflows and outflows.

Junctions are usually used to represent stream confluence points. In this study Junctions are also used to represent the final discharge from the study area, and to summarize sub-basin flows with hydraulic element flows.

Junction elements are also used to represent the final flow from sub-basins which are not connected to the main flow network. These final flow sub-basins are numbers 16, 17, 18 which are considered minor drainages rather than major drainages. As noted earlier, detailed analysis of minor drainages was beyond the scope of the project; however, some level of discussion was warranted for each of these minor sub-basins as provided in Section 10.

## 9.2 SUMMARY RESULTS

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for existing land use conditions existing hydraulic conditions are given on Table 5 on Page 30 at the rear of Section 6.

Summarized results of the Hydraulic calculations and routing for the 2 Year, 10 Year, and 100 Year events for future land use conditions and the proposed future DBDPA hydraulic conditions are given on Table 6 on Page 31 at the rear of Section 6. These values are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

Summarized results that are direct printouts from the HMS run are also included in Appendix A for existing land use and existing hydraulic conditions.

Summarized results that are direct printouts from the HMS run are also included in Appendix B for future land use and future hydraulic conditions. These printouts are the fully implemented DBDPA as proposed in this report and are the values used whenever reference is made to DBDPA flows.

A summary of the recommended improvements is found is Section 13 of this report and is entitled "Major Recommendations Summary, Cost Estimate, and Prioritization."

Figures 16, 17, and 18 are enlarged site plan drawings illustrating the areas where major recommendations are proposed.

Hydrographs of the DBDPA condition flow elements, detention ponds, and junctions are included in Appendix C.

## 9.3 DETAILED JUNCTION ELEMENT DISCUSSION

No detailed discussion is provided for the Junction Elements due to the nature of the element. Rather the only information provided is a general description of the Junction Element location.

Refer to Section 7 for Channel and Pipe Elements, Section 8 for Detention Ponds, and Section 10 for modeled “minor” basins.

### JUNCTION 201

Junction 201 is located upstream of Homestead Street and is in Detention Pond 104. This junction provides the inflow hydrograph to Detention Pond 104 by summarizing flows from Sub-basin 2 and Element 2.

### JUNCTION 202

Junction 202 is located downsteam of Homestead Street and is in Detention Pond 101. This junction provides the inflow hydrograph to Detention Pond 101 by summarizing flows from Sub-basin 3 and Element 4.

### JUNCTION 203

Junction 203 is located in Detention Pond 100. This junction provides the inflow hydrograph to Detention Pond 100 by summarizing flows from Sub-basin 6 and Element 8.

### JUNCTION 204

Junction 204 is located just downstream of Plateau Lane. This junction provides the inflow hydrograph to Element 12 by summarizing flows from Sub-basin 7 and Element 11.

### JUNCTION 205

Junction 205 is located upstream of Twilight Drive. This junction provides the inflow hydrograph to Junction 206 by summarizing flows from Sub-basin 4 and Element 7.

### JUNCTION 206

Junction 206 is located at the upstream face of Twilight Drive. This junction provides the inflow hydrograph to Element 13 by summarizing flows from Junction 205 and Element 12.

### JUNCTION 207

Junction 207 is located upstream of Albert Lane at the inlet to Element 15. This junction provides the inflow hydrograph to Element 15 by summarizing flows from Sub-basin 8E and Element 14.

### JUNCTION 208

Junction 208 is located downstream of Albert Lane at the outlet from Element 15. This junction provides the inflow hydrograph to Element 16 by summarizing flows from Sub-basin 8W and Element 15.

**JUNCTION 209**

Junction 209 is located upstream of Reservoir Road. This junction provides the inflow hydrograph to Element 17 by summarizing flows from Element 16 and Element 23. In this case it is noted that the Junction hydrograph is not a true representation of all flows because a portion of Sub-basin 10 also drains to this area but is not accounted for by the Junction. Refer to Element 17 for the actual estimated flow at this location.

**JUNCTION 210**

Junction 210 is located at the upstream face of Longview Road. This junction provides the inflow hydrograph to Element 18 by summarizing flows from Sub-basin 10 and Element 17A.

**JUNCTION 211**

Junction 211 is located at the upstream face of Highway 44. This junction provides the inflow hydrograph to Element 20 by summarizing flows from Sub-basin 11 and Element 19. In this case it is noted that the Junction hydrograph is not a true representation of all flows because a portion of Sub-basin 11 may actually bypass the Junction or be located on the south side of Highway 44. Refer to Elements 19 and 20 for additional discussion.

**JUNCTION 212**

Junction 212 is located at the downstream face of Highway 44. This junction provides the inflow hydrograph to Element 21 by summarizing flows from Element 20 and Element 53.

**JUNCTION 213**

Junction 213 is located where Element 22 drains into Rapid Creek. Junction 213 is in the Rapid Creek floodway. This junction provides the final discharge from study area less Sub-basins 16, 17, 18 which are described elsewhere.

**JUNCTION 250**

Junction 250 is located downstream of Longview Road. This junction provides the inflow hydrograph to Detention Pond 105 by summarizing flows from Sub-basin 15 and Element 50.

**JUNCTION 251**

Junction 250 is located at the intersection of Highway 44 and Reservoir Road. This junction provides the inflow hydrograph to Element 53 by summarizing flows from Sub-basin 12 and Element 52.

**JUNCTION 260**

Junction 260 is located at the mouth of Sub-basin 16. Junction 260 is in the Rapid Creek floodway. This junction is a repeat of the Sub-basin 16 hydrograph. Because Sub-basin 16 is considered a Minor Study area Junction 260 is used for graphical purposes rather than hydrograph summation.

## JUNCTION 261

Junction 261 is located at the mouth of Sub-basin 17. Junction 261 is in the Rapid Creek floodway. This junction is a repeat of the Sub-basin 17 hydrograph. Because Sub-basin 17 is considered a Minor Study area Junction 261 is used for graphical purposes rather than hydrograph summation.

## JUNCTION 262

Junction 262 is located at the mouth of Sub-basin 18. Junction 262 is in the Rapid Creek floodway. This junction is a repeat of the Sub-basin 18 hydrograph. Because Sub-basin 18 is considered a Minor Study area Junction 262 is used for graphical purposes rather than hydrograph summation.

(End of Section 9 Narrative)

# 10. MINOR SUB-BASINS

This DBDPA provides for only major drainage. Unless specifically noted in the study, analysis of localized or minor drainage and/or minor sub-basins was beyond the scope of work.

Sub basins 16, 17, and 18 were included in the study even though they are not connected to the main routing system that conveys flows for Sub-basins 1 – 15. These 3 sub-basins are considered minor sub-basins and thus detailed study was beyond the scope of work. Nevertheless a certain level of discussion is warranted for each of these sub-basins as follows.

These 3 minor sub-basins discharge to Rapid Creek in the FEMA designated floodway.

**SUB-BASIN 16**

Sub-basin 16 has a significant amount of existing development. Almost all of the existing development is on the north side of Highway 44.

Based on area prorating the fully developed flows on the north side of Highway 44 are:

2 Year = 30 cfs
10 Year = 110 cfs
100 Year = 215 cfs

Based on the same area prorating the existing condition flows on the north side of Highway 44 are:

2 Year = 20 cfs
10 Year = 75cfs
100 Year = 185 cfs

Two pipes are currently in place to convey flows from the north side of Highway 44 to the south side of Highway 44.

One of the Highway 44 crossings is a dual pipe system located just east of Teewinot Drive. Only a portion of the basin north of Highway 44 reaches this pipe. Based on original design plans it is estimated this crossing has capacity for about 65 cfs. This pipe and inlet system was installed by private parties with the approval of the SDDOT and the City of Rapid City. Flows that bypass this pipe will continue east in the Highway 44 ditch. Some of these bypass flows may also enter the Murphy Irrigation Ditch.

Flows that bypass this dual pipe system plus the remaining basin flow from north of Highway 44 reaches the second Highway 44 cross pipe which is located about 300’ east of Longview Road. A ditch block is located about 500’ east of this cross pipe. This block increases the head available for the cross pipe and will also cause some detention to occur in the nearby area. This Highway 44 pipe has capacity for only about 25 cfs.

Combined capacity of the Highway 44 crossings is thus about 90 cfs which is less than the developed condition 10 year flow but exceeds the existing condition 10 year flow. Because the crossings have capacity for the existing 10 year storm it is recommended that all future projects, including redevelopments, in the basin north of Highway 44 be required to maintain runoff conditions to existing conditions or less. By doing so it is estimated that the pipes under Highway 44 will be adequate to convey the 10 Year storm without roadway encroachment or overtopping. Optionally, an additional culvert could be installed under Highway 44 so the developed 10 year flows can be conveyed under the Highway.

Some of the basin is on the north side of the Murphy Irrigation Ditch. Diversion structure(s) should be installed during future developments to divert stormwater flows from the ditch to the Highway 44 ditch and pipe system.

Driveway and roadway crossings of the Highway 44 north ditch provide some additional minor but unaccounted for storm water detention.

Detailed design analysis of the basin is needed at the time future projects occur. As described above any future projects/redevelopments in the basin need to have small detention ponds to maintain flows to existing conditions unless detailed analysis indicates otherwise. It noted that the developed discharge for this sub-basin ignores the effects of any existing or future minor detention ponds.

Channels on the south side of the highway will need to be increased in size as necessary as development occurs. Under current conditions these small channels may overtop onto the adjacent farmland and/or wetlands.

## SUB-BASIN 17

Sub-basin 17 is entirely on the south side of Highway 44. Other than a few agricultural buildings it is devoid of development.

During development it will be necessary to construct onsite detention ponds to meet the RCIDCM requirements of maintaining flow rates to existing conditions and to meet the water quality requirements. The developed DBDPA flows given in this report ignore the effects of these small ponds.

Drainage channels will need to be constructed as necessary during development.

The mouth of the drainage basin is near the diversion structure for the Little Giant Irrigation Ditch. Provisions must be made during development so stormwater flows do not enter the ditch.

## SUB-BASIN 18

Sub-basin 18 is entirely on the south side of Highway 44. Other than a few agricultural buildings it is devoid of development.

During development it will be necessary to construct onsite detention ponds to meet the RCIDCM requirements of maintaining flow rates to existing conditions and to meet the water quality requirements. The developed DBDPA flows given in this report ignore the effects of these small ponds.

Drainage channels will need to be constructed as necessary during development.

The Little Giant Irrigation Ditch crosses this sub-basin. Provisions must be made during development to prevent developed stormwater flows from entering the ditch and/or to provide diversions out of the ditch for flows that may enter.

# 11. IRRIGATION DITCHES

## 11.1 BACKGROUND

Three main irrigation ditches are located in the study area. These ditches are the Hawthorne Ditch, Murphy Ditch, and the Little Giant Ditch. The ditches are indicated on Figure 1.

Each ditch has a specific water right. SDDENR provided the following information regarding the water rights for each ditch.

- Hawthorne Ditch: WR# 2039-2 19.98 cfs 986 acres
- Murphy Ditch: WR#1727-2 7.25 cfs 381.1 acres
- Little Giant Ditch: WR#2383-2 5.26 cfs, 283.9 acres

Descriptions and recommendations for each of the ditches follows. It is noted that the respective ditch company will need to be contacted per the RCIDCM requirements for approval of any work that affects the ditch or that results in additional stormwater flows into the ditch.

## 11.2 HAWTHORNE DITCH

### 11.2.1 DESCRIPTION OF HAWTHORNE DITCH

The Hawthorne Ditch is the largest of the ditches in the study area. It enters the study area near the intersection of Covington Street and Haven Street and exits the study are just east of Reservoir Road. It flows east though the study area. Storm water runoff from Sub-basin 8W and the lower 35%+- of Sub-basin 9 is directly intercepted by the ditch.

A 30" RCP storm sewer along Covington Street discharges directly into the ditch on the east side of the street. The pipe discharges into the ditch on the north side of Haven Street at Covington Street. It is our understanding the ditch company allowed this direct discharge in return for the ditch being placed in a pipe from just west of Sweetbriar Street to the east side of Covington Street.

Another small storm sewer system discharges directly into the ditch at Sprucewood Street. Other surface drainage enters the ditch from the north more or less along the full length of the ditch.

Following is a list of the existing irrigation pipe/structures at each location where a roadway crosses the ditch.

- Covington Street – 42" RCP irrigation pipeline that begins just west of Sweetbriar Street and ends just east of Covington Street.
- Reed Court – 54" Arch RCP culvert.
- Plateau Lane – 60" Arch CMP culvert
- Sprucewood Street – Twin 42" Arch RCP culverts
- Reservoir Road – 7' x 3' Concrete Box Culvert

Element 23, a storm sewer, carries the upstream portion of Sub-basin 9 over the ditch at Reservoir Road.

Element 15 carries the upstream sub-basin flows over the ditch. A 30" CMP inverted siphon on the ditch is located under Element 15 just upstream of box culvert outlet. Original plans for the siphon indicate a 50' long overflow was to be constructed at this location; however, the overflow was not evident in the field due to debris and vegetation in the area. There is also a waste gate at this location with stop logs that provide some small overflow relief and allows the ditch to be completely diverted to the downstream major drainage channel. The waste gate is not watertight and allows leakage into the downstream channel. Based on a review of the design drawings it appears the intent of the structure is that overtopping to the main storm drainage channel system will begin when flows exceed 20 cfs.

An overflow structure exists near the upstream end of the Reservoir Road crossing. This is a cast in place manhole structure with a stop log assembly. Based on a review of the design drawings it appears the intent of the structure is that overtopping into the overflow manhole will begin when flows exceed 20 cfs. With the stop logs in place the overtopping capacity is limited to a 4' wide by 8" high opening. The stop logs can be removed if diversion of the entire ditch to the downstream storm sewer is desired. Overflow into the diversion structure was observed when the only flows present were irrigation flows. It was beyond the scope of the study to investigate why this was occurring although logical reasons may be (1) backwater created by downstream ditch blockage (2) irrigation flow at a rate higher than designed for, (3) diversion and roadway crossing pipe not constructed to plan grades.

Heavy vegetation was observed along the banks of the ditch in several locations. There are a few locations where trees are actually in the ditch bottom. Neighboring property owners are also using the ditch as a location to dispose of animal waste, grass clippings, tree trimmings, and other such objectionable materials. This material has a known history of plugging trash racks and pipes which leads to flooding issues.

Representatives of the irrigation ditch company verbally verified problems along the ditch as noted above. They noted problems related to plugging of the roadway crossings with related flooding, ditch overtopping, leaking waste gates, overtopping into the overflow structure at Reservoir Road during normal irrigation flows, and neighbors using the ditch as a dumping ground.

## 11.2.2 IMPROVEMENT RECOMMENDATIONS FOR HAWTHORNE DITCH

Improvements to the Hawthorne Ditch, including recommendations for new adjacent storm sewers, are recommended as described below. Figure 19 illustrates the recommended improvements.

It is noted that the following recommendations are based on use of historic drawings and general field observations and measurements. No field surveys along the Hawthorne Ditch were made. It is necessary that a detailed survey of the entire ditch be made and recommendations verified against the survey and final design. The survey should include verification of water elevations when irrigation flows are at the maximum allowed. The final design should include a detailed HECRAS analysis of the entire system. The final design should also take into account whether or not downstream modifications can be made to reduce flow depth at Reservoir Road which then translates to upstream areas.

### 11.2.2.1 - COVINGTON STREET TO ELEMENT 16

*(Note: Pennington County constructed a portion of the following recommendations in 2013 while the draft of this report was in review by the City of Rapid City). Plateau Lane was reconstructed with curb and gutter and storm sewer. The storm sewer connects directly to the Hawthorne Ditch. The flow diversion recommendations at the Hawthorne Ditch as discussed below were not included with that project.)*

A primary component of the recommendations is redirect Sub-basin 8W flows. This recommendation requires a storm sewer along Plateau Lane north of the ditch. Pennington County verbally indicated they are proposing to reconstruct Plateau Lane in the foreseeable future and that the project will include a storm sewer.

In regards to this storm sewer improvement it is understood Pennington County is planning a street with curb and gutter and storm sewer. The following discussion is based on the assumption of curb and gutter and storm sewer; however, it is recommended Pennington County use ditches and swales if possible for the storm conveyance system as a Post Construction Storm Water Quality Management technique.

It is recommended the Plateau Lane storm drainage system extend north of the Hawthorne Ditch to at least Howie Drive. Extending the pipe this far will allow future development/redevelopments on each side of the street to connect their drainage systems into the pipe. The storm sewer needs to be installed deep enough to allow the area between Plateau Lane and Covington Street to be drained east to the new pipe as indicated on Figure 19.

Based on area prorating the 100 year flow to the proposed Plateau Lane system is about 62 cfs. It is recommended the Plateau Lane storm sewer be sized for a minimum of 45 cfs from the Hawthorne Ditch north to Roberts Court and then a minimum of 35 cfs from Roberts Court to Leroy Street. The pipe can be progressively downsized between Leroy Street and Howie Drive.

The flows given above are for the 10 year flows from the east side of the street plus the 100 year redirected flow from the west side of the street. It is estimated a 42” Arch RCP is needed for 45 cfs, a 36” RCP is needed for 35 cfs, and then the pipe will progressively decrease to an 18” RCP at the upstream end. The remainder of the flow will have to be carried on the street or in a ditch. Final pipe sizing is necessary as part of the final design.

The storm sewer should turn east at Plateau Lane. Additional inlet and pipe capacity is needed at this location for the remaining 17 cfs (62 cfs -45 cfs) that is not captured by the Plateau Lane Storm Sewer. The storm sewer should extend about 150’ to discharge east of the lot that is on the south side of the ditch. From that point a channel should be graded on the south side of the ditch to convey the storm sewer flows to Element 16.

The storm sewer running east of Plateau Lane should be sized for 98 cfs which is the entire 100 year flow from Sub-basin 8W. It is estimated that the available slope will be 0.5% which results in a required pipe size of 54” Arch RCP for the 98 cfs.

It will be necessary to verify the irrigation ditch is “watertight” to prevent exfiltration from the ditch to the proposed parallel graded storm channel. Lining of the ditch with clay, HDPE, or other suitable material may be required to prevent the exfiltration. Optionally the ditch could be completely enclosed with a pipe sized for only 20 cfs as a result of the diverted storm flows. At an invert slope of 0.0015 ft/ft a 36” RCP would carry 20 cfs at a depth of about 2’. Another option would be to extend the 54” RCP to Element 16.

Installation of the above storm sewer system and redirection of west side flows is estimated to reduce the 100 year storm flow reaching the ditch west of Plateau Lane to about 36 cfs. This flow was calculated by area prorating of the Sub-basin 8W flow accounting for the redirected area to the Plateau Lane storm sewer. Adding the 20 cfs irrigation flow results in 56 cfs in the irrigation ditch west of Plateau Lane. An overflow weir should be installed on the west side of Plateau lane to direct the 36 cfs storm flow to the proposed storm sewer resulting in the total storm sewer capacity of 98 cfs as recommended above.

An irrigation ditch siphon is needed at Plateau Lane to allow the storm sewer to be on the south side of the ditch. The siphon should be sized for a minimum of 20 cfs *(irrigation flow only)* based on the recommendation that all storm flows be directed to the new storm sewer. However, during final design it will be necessary to make a determination as to whether the siphon, or storm sewer, should be oversized to account for uncertainties regarding when full implementation of all recommendations and the assumed West Basin Transfer will occur.

An irrigation ditch pipe will likely have to be continued downstream of the siphon the same distance as the storm sewer due to space limitations for the storm sewer installation.

Normal depth calculations indicate 56 cfs can be carried in the segment of ditch upstream of Plateau Lane. The normal depth calculations assume a trapezoidal channel with a 6' bottom, 1.5H:1V side slopes, invert slope of 0.0015, and n value of 0.035. Normal depth would be 2.5' compared to an estimated available channel depth of about 4'.

It is recommended that the Reed Court pipe be replaced with a 7' x 3' concrete box culvert to reduce backwater that may occur as a result of storm flows.

Improvements to the existing siphon at Element 16 are also recommended. The leaking waste gate should be made watertight. This is intended to prevent leaks which otherwise create a loss of irrigation water and contribute to undesired based flows in the channels. As an option, a different type of gate such as a watertight slide gate could be considered.

The overflow at this existing siphon should also be regraded and provided with a "hardened" weir so the shape remains. The overflow is recommended as a redundant safety factor to divert flows that may exceed the 20 cfs irrigation flow.

If the irrigation company objects to a siphon at Plateau Lane it will likely be necessary to discharge all storm flows into the irrigation ditch and provide a lined overflow structure downstream of Plateau Lane. In this case the existing pipe under Plateau Lane will need to be increased to convey 56 cfs, the channel *(irrigation ditch*) downstream of Plateau Lane increased in size for 118 cfs, and the linear overflow structure east of Plateau Lane sized to discharge 98 cfs over the south bank as quickly as possible. The linear overflow would discharge into a separate storm channel that drains to Element 16 similar to the channel required by the storm sewer recommended above.

#### 11.2.2.2 - ELEMENT 16 TO RESERVOIR ROAD

Improvements recommended west of Element 16 reduce the flows to this segment of the ditch. Ditch flows are thus estimated at 20 cfs *(irrigation flow only)* at the west end of this segment *(at the discharge from the Element 16 siphon).* Flow in the ditch at Reservoir Road is estimated at 80 cfs *(20 cfs irrigation flow plus 60 cfs area prorated from Sub-basin 9.)* The majority of the storm flow enters towards the downstream end of the ditch.

Normal depth calculations indicate 80 cfs can be carried in the ditch. The normal depth calculations assume a trapezoidal channel with a 6' bottom, 1.5H:1V side slopes, invert slope of 0.0015, and n value of 0.035. Normal depth would be 3.0'. It is assumed 4' of channel depth is available.

Flow depth at the inlet end of the Reservoir Road irrigation ditch box culvert was measured when only irrigation flows were present. The measured flow depth was about 2.35'. The flow depth is obviously being controlled by the depth of flow in the downstream channel. It was beyond the scope of this study to make recommendations to reduce the flow depth on the downstream side of the box culvert, although it is obvious any decrease in downstream flow depth would be beneficial in reducing upstream flow depth.

It is recommended that the existing overflow/waste gate structure at Reservoir Road be replaced with a structure that will direct all flows in excess of 20 cfs to the Element 23 storm sewer system. The existing structure has only minor capacity for overflows.

Based on an assumption that the maximum overflow depth should not exceed 0.65' in depth *(difference between normal depth for 80 cfs and measured inlet depth for 20 cfs)*, the new overflow structure weir should be a minimum of 40' in length. The overflow structure would discharge into the Element 23 storm sewer.

The new overflow structure should also have a waste gate so the entire flow can be directed to the storm sewer system if necessary. The waste gate should be watertight.

The Sprucewood Street crossing of the ditch was evaluated using HY8 with an assumed tailwater depth of 3'. Only minor additional flows should enter the ditch upstream of this crossing; nevertheless, 40 cfs was used for the crossing evaluation rather that the 20 cfs irrigation flow. The HY8 model indicates the culverts will pass 40 cfs with about 9" of freeboard available.

## 11.3 MURPHY DITCH

### 11.3.1 DESCRIPTION OF MURPHY DITCH

The Murphy Ditch enters the study area near the intersection of Highway 44 and Covington Street and exits the study area just east of Element 18. It flows east though the study area. Storm water runoff from parts of Sub-basins 11, 12, and 16 enter the ditch. The discharge from Detention Pond 105 and thus Sub-basins 14 and 15 also enters the ditch.

An 18" CMP storm sewer discharges directly into the ditch on the east side of Reservoir Road. A 10" steel pipe from Detention Pond 105 currently discharges directly into the ditch.

Following is a list of the irrigation pipe/structures at each location where a roadway crosses the ditch.

- Highway 44 and Teewinot Drive – 36" HDPE irrigation pipeline beginning on south side of Highway 44 and ending just east of Teewinot Drive.
- Longview Road – 48" RCP culvert
- Private Driveway west of Reservoir Road connected to Highway 44 – "Homemade" Bridge/Culvert structure
- Reservoir Road – 48" RCP Culvert

A 36" CMP inverted siphon carries the ditch under the main drainage channel at the downstream end of Element 18. This is about 900' east of Reservoir Road. Original plans for the siphon indicate a 50' long graded overflow was to be constructed at this location. There is also a waste gate at this location with stop logs that provide some small overflow relief and allow the ditch to be completely diverted to the downstream major drainage channel by removal of the stop logs. The waste gate is not of watertight design. Based on a review of the design drawings it appears the intent of the structure is that overtopping to the channel system will begin when flows exceed about 14 cfs.

Overflow diversion to the south can occur at the upstream end of the Reservoir Road crossing. There is not an engineered device with any specific overflow capability.

Heavy vegetation was observed along the banks of the ditch in several locations. Neighboring property owners are also using the ditch as a location to dispose of animal waste, grass clippings, tree trimmings, and other such objectionable materials. This material can causes plugging of racks and pipes which can lead to flooding issues.

A representative of the ditch company verbally indicated they have not experienced any significant problems related to storm water entering the ditch. However they commented that as development continues to occur they expect appropriate design measures will be required.

### 11.3.2 IMPROVEMENT RECOMMENDATIONS FOR MURPHY DITCH

Recommended improvements to the Murphy Ditch are as follows.

The leaking waste gate at the siphon east of Reservoir Road should be made watertight. This is intended to prevent leaks which otherwise create a loss of irrigation water and contribute to undesired based flows in the channels.

The existing graded Diversion Structure at the siphon will need to be adjusted as necessary when improvements to Elements 18 and 19 are made.

Recommendations for Sub-basin 16, a minor drainage area, include construction of diversions from the ditch so that flows in excess of the irrigation flow are directed to the Highway 44 drainage system. This is discussed in Section 10.

Recommendations for Element 52 include piping the storm water flow over the ditch. An alternate may be to use an Irrigation Siphon under Element 52. It is recommended that trickle flows/minor flows be allowed to enter the ditch, if allowed by the ditch company, for water quality purposes.

An Engineered Diversion Structure should be constructed at the upstream end of the Reservoir Road crossing when development occurs in Sub-basin 12. The design should be coordinated with the design of Detention Pond 107. The diverted stormwater should be directed to Detention Pond 107. Final capacity of the Diversion Structure is dependent on how Basin 12 actually develops and whether or not diversions on the ditch upstream of this location have been installed.

The Private Driveway crossing should be replaced with an Engineered culvert system at such time as the existing structure needs replacement or at such time as Sub-basin 12 is developed. Capacity of the culvert should be as required to pass the 7.25 cfs design flow with no backwater affects. The structure should be oversized as necessary and approved by the Ditch Company to minimize plugging. A diversion should be created at this same location to divert excess flow to the Element 52 channel.

It is anticipated that future development in Sub-basin 11 north of the ditch will be graded to drain to Element 18. If this is not the case it will be necessary to intercept the flow and carry it over the ditch or otherwise construct diversion structures to divert excess flows back out of the ditch to an appropriate drainage system.

## 11.4 LITTLE GIANT DITCH

The creek diversion structure for the Little Giant Ditch is located about 1500' west of Reservoir Road. The ditch exits the study area at the east side of Element 22. It flows east though the study area. Storm water runoff from parts of Sub-basins 13 and 18 enter the ditch as surface flows.

There are no known storm sewer discharges into this ditch. Runoff that reaches the ditch is almost entirely from undeveloped land.

The only road crossing is the culvert under Reservoir Road.

A 36" CMP inverted siphon carries the ditch under the main drainage channel identified as Element 22. Original plans for the siphon do not show any overflow device at this siphon. A waste gate with stop logs was constructed at this location. The stop logs provide some small overflow relief and allows the ditch to be completely diverted to the downstream major drainage channel. Based on a review of the design drawings it appears the siphon has capacity that exceeds the 5.3 cfs water right.

Heavy vegetation was observed along the banks of the ditch in several locations.

The waste gate at the siphon leaks and should be repaired with a watertight system.

It is believed there are no current flooding problems along the ditch. However, flooding issues could arise when urbanization occurs along the ditch. Proper design is needed when development occurs in the upstream basins to (1) prevent developed area flows from entering the ditch, (2) to create overflow capacity as necessary, or (3) to improve ditch capacity to convey storm flows.

# 12. STORM WATER QUALITY

## 12.1 BACKGROUND

In 1987, Congress amended the federal Clean Water Act to require implementation, in two phases, of a comprehensive national program for addressing storm water discharges. The first phase of the program, commonly referred to as "Phase I," was promulgated on November 16, 1990 but does not currently apply to either the City of Rapid City or Pennington County.

On December 8, 1999, EPA promulgated "Phase II" of the Storm Water Regulations, which expanded the program to include point source discharges from Small Municipal Separate Storm Sewer Systems (MS4). The City of Rapid City and the urban area of Pennington County are each designated as an MS4 and as such are subject to the requirements of the "Phase II" regulations. SDDOT by definition is also an MS4.

The South Dakota Department of Environment and Natural Resources (DENR) has been the delegated permitting authority for the Storm Water Program within the State of South Dakota since December 1993, and has adopted the federal storm water regulation by implementing the South Dakota General Permit for Storm Water Discharge from Small Municipal Storm Sewer Systems.

State of South Dakota regulations require the MS4 to "develop, implement, and enforce a storm water management program designed to reduce the discharge of pollutants from the MS4 to the Maximum Extent Practicable to protect water quality." In short, the MS4 must develop procedures that meet the requirements of following six minimum measures and protect waters of the state from pollution, contamination, and/or degradation. The six minimum measures are:

1) Public education and outreach;
2) Public participation/involvement;
3) Illicit discharge detection and elimination;
4) Construction site storm water runoff control;
5) Post-construction storm water management; and,
6) Pollution prevention/good housekeeping for municipal operations.

For the purposes of this report only Post Construction Storm Water Management is discussed. The City of Rapid City and Pennington County have ordinances and design/guidance manuals dealing with storm water quality as required by the Phase II rules and the South Dakota General Permit. Both agencies have requirements that require controls to prevent or minimize water quality impacts.

It is beyond the scope of this report to provide detailed design or recommendations for the Post Construction Storm Water Management Controls. Rather the following recommendations are based on (1) regional type devices that can be incorporated into future projects and (2) other miscellaneous discussion considered applicable to water quality in the study area.

## 12.2 WATER QUALITY TREATMENT AT DETENTION PONDS

Detention ponds provide locations for storm water quality treatment. Treatment methods than can be used within detention ponds include but are not necessarily limited to:

- Extended Detention
- Wetland Bottoms
- Retention (Lake)
- Wetland Channels in Pond Bottom
- Filtering Devices at Outlet Structures

It is recommended that Stormwater Treatment be incorporated at all detention ponds described in the report and as discussed below.

### 12.2.1 DETENTION PONDS 100, 103, 106, AND 107

Extended detention is recommended for Detention Ponds 100, 103, 106, and 107. The stage storage discharge curves in Section 8 include allowance for Extended Detention storage. Extended Detention storage allowance was approximated using the 40 hour Drain Time curve in the City of Rapid City Stormwater Quality Manual.

Only the local basin to each pond was used for estimation of Extended Detention storage sizing.

The Extended Detention discharge orifice described in Section 8 for these ponds is considered a “dummy” size simply for HMS modeling.

Final sizing of the Extended Detention storage and discharge characteristics is required during final design of the pond improvements. Final storage requirements should consider if credits can be taken for any upstream “Stormwater Better Site Design and Techniques.” In some cases it may not be possible to provide 40 hour storage volume and Engineering judgement will be necessary for the sizing.

### 12.2.2 DETENTION POND 101

Detention Pond 101 cannot be graded to create Extended Detention because of its small size, restricted work area, and wetlands.

The City of Rapid City noted the original design and construction of Pond 101 required a level of approval by the USCOE due to wetlands. The City noted this is why the pond spillway is not in the location indicated on the original subdivision drawings. Because of the USCOE issue, the City stated that any improvements to this pond were not to any include regrading of the pond bottom, additional pipes, or relocation of the pipes.

Nevertheless, Stormwater Quality Treatment can be incorporated into the design.

Wetland vegetation is starting to emerge in the bottom of the pond. The City noted that local residents were aware this pond may become a wetland bottom when it was constructed. It is recommended that the pond bottom continue to be allowed to take on wetland characteristics for water quality issues. A wetland bottom will have no effect on the detention characteristics of the pond.

If allowed by USCOE, it is recommended the pond bottom be regraded to enhance the wetland treatment. This would consist of shaping a small meandering low flow channel in the pond bottom. The channel would have micropools for settling/dilution of sediments and to provide habitat for species that prey on mosquitos. This would also allow drying of the flatter pond bottom areas to allow for maintenance.

Recommendations for the pond also include the installation of a riser pipe with orifices for flow control. Trash racks are recommended for the orifices. It is recommended that a granular filter be installed in front of the trash rack on the lowest orifice to enhance sedimentation.

### 12.2.3 DETENTION POND 104

Detention Pond 104 cannot be graded to create Extended Detention because of its small size and restricted work area. Nevertheless, Stormwater Quality Treatment can be incorporated into the design.

Recommendations for this pond include regrading to provide a defined low flow channel. It is expected that the low flow channel grading will result in a flat longitudinal slope which overtime will become a linear wetland water quality feature.

Recommendations for the pond also include the installation of a riser pipe with orifices for flow control. Trash racks are recommended for the orifices. It is recommended that a granular filter be installed in front of the trash rack on the lowest orifice to enhance sedimentation.

### 12.2.4 DETENTION PONDS 102 AND 105

Detention Ponds 102 and 105 are existing permanent pool "lakes" and also serve as detention ponds.

Stormwater quality in these ponds is provided by the permanent pool lakes which are considered as "stormwater quality retention ponds." The lakes are also surrounded by wetland type vegetation which provides additional treatment.

Detention recommendations for these ponds include modifications of the outlets structures. The low flow portions of the structures should be designed using appropriate design techniques for wet ponds such as hoods, underwater inlets, and maximizing detention time for low flow events.

## 12.3 ON-SITE WATER QUALITY TREATMENT

Recommendations for storm water quality treatment locations and methods outlined above do not preclude the requirement that individual development/redevelopment projects abide by City and County rules and regulations for Post Construction Storm Water Treatment.

The City regulations include a BMP Evaluation Form that takes into account Regional BMP's. For the purposes of this study it is recommended the only Regional BMP's to be considered are Detention Ponds 100, 102, 103, 105, 106, and107. However, the ponds should not be considered as being an available regional BMP until such time as the recommended improvements are constructed.

An option that can be considered is allowing Developers to make improvements to adjacent regional Detention Ponds in lieu of the otherwise required onsite treatment. In this case Developers would not lose valuable land, would apply their onsite funding to needed regional projects, and the City would receive regional improvements at no cost.

It is also recommended that all developments be required to use proper topsoil and amendments, including thickness, to promote infiltration of direct precipitation on pervious surface and to improve infiltration of discharges from impervious surfaces that drain over pervious area. In addition the soils should be loosened if they have been compacted prior to seeding.

## 12.4 WATER QUALITY TREATMENT IN CHANNELS

A certain level of stormwater quality treatment is provided by vegetated channels. Wetland vegetation in channel bottoms improves treatment by enhancement of sedimentation and by biological uptake of nutrients.

Stable, flat grade channels will slow water down to promote settling and discourage headcut erosion. The majority of the existing manmade channels in the project area have flat grades and a certain amount of sediment deposition is evident.

Steep natural channels that will be required to carry frequent urban flows need to be evaluated carefully for stability. The frequent urban runoff will commonly lead to channel instability and subsequent severe erosion even if the channels appear stable under historic conditions. Channel erosion such as this is a leading contributor to storm water pollution. In most cases it will be necessary to add channel stabilization devices as part of the development or otherwise regrade the channel to a stable geometry.

Future engineered channels should be designed to promote slow flows and to prevent headcutting. The individual Element discussions include these types of recommendations for various channels.

Future engineered channels or channels that are recommended for reconstruction should also incorporate composite channel design where possible. Composite channel design will allow the low flow channel to take on wetland characteristics while allowing the “overbank” areas to be maintained.

Specifically it is recommended that channel Elements 17, 17A, 18, 19, 20, 21 and 22 incorporate composite channel design for water quality enhancement.

Final design of the wetland low flow portion of any composite channel should consider micropools for sedimentation areas and to create habitat for species that prey on mosquitos. It is also expected many of the proposed composite channels will be constructed by developers as part of adjacent subdivision projects. In those cases it may be possible to incorporate additional or larger treatment measures *(expanded or off line wetlands, pools, ponds, sediment filters, etc)* adjacent to or in the channel to provide the required on-site Post Construction Pollution Control.

Channel Element 2 is also recommended to be constructed at a flatter grade with either a low flow or trickle channel.

## 12.5 WATER QUALITY TREATMENT IN WETLANDS

In addition to the Wetlands in Detention Ponds and Channels described above, the potential exists that other wetlands may exist in the study area as described in Section 4 and shown on Figure 9.

Properly managed wetlands can intercept runoff and treat storm water pollutants such as sediment, nutrients, and certain heavy metals. Wetland vegetation also helps channel stability by slowing runoff and by evenly distributing the energy in runoff. Wetland vegetation can also cool stream temperature by providing shade.

Wetlands can be impaired by improper development or excessive pollutant loads. Impaired or degraded wetlands may not provide water quality treatment and can actually become pollutant sources. Degraded wetlands can release decaying vegetation, stored nutrients and other chemicals into surface water and ground water.

Proper use of wetlands generally includes the following three strategies.

- Preserve wetlands and prevent their degradation.
- Restore impaired wetlands.
- Pretreat runoff before it reaches wetlands

It is recommended the above wetland strategies by implemented to the maximum extent practicable and in no case should USCOE wetland regulations be violated.

## 12.6 WATER QUALITY TREATMENT PROVIDED BY IRRIGATION DITCHES

Three main irrigation ditches cross the study area as discussed earlier. Opportunity exists to utilize these ditches for storm water quality treatment.

Stormwater flows enter the ditches at various locations. These flows are then diluted by the “clean” irrigation water and also conveyed downstream to be treated by land application on crops. A certain amount of sedimentation also occurs in the ditches due to low flow. Trash racks on the ditch also capture larger debris type pollutants.

This DBDPA recommends stormwater flows be diverted from the ditches to the main stormwater conveyance system. However, in many instances flows will still enter the ditches before being diverted back out of the ditch.

In locations where main conveyances cross the ditches it is recommended the ditch company be contacted to allow minor flows to enter the ditches for water quality purposes. If this type of proposal is acceptable to the respective ditch company it will be necessary to size the system so the ditch capacity is not compromised.

## 12.7 WATER QUALITY TREATMENT WITH ROADWAY PROJECTS

The City of Rapid City, Pennington County, and SDDOT will all be involved with roadway construction/reconstruction projects in the study area at some time. Post Construction Storm Water Management Control is required of all three agencies by the SDDENR permit and by their own ordinances and manuals.

Roadway construction/reconstruction projects need to be reviewed to determine if the scope of the project requires that Post Construction Storm Water Management measures be implemented. Designers need to be made aware that an Erosion and Sediment Control plan and related Stormwater Pollution Protection Plan are for the construction period only and are not by themselves considered Post Construction controls. Many street reconstruction projects in this area are expected to warrant Post Construction controls because of the nature of the existing street system.

It is recommended that the City, County, and SDDOT be leaders in the use of LID techniques for public projects including roadway reconstruction projects. This type of construction incorporates such things as drainage swales/ditches rather than curb and gutter, narrow pavements, green center medians rather than paved medians, green islands in cul-de-sacs, pavement edge biofilters, street tree plantings, porous pavements, etc. These types of installation are considered Post Construction controls which the agencies are required to implement on their own facilities.

## 12.8 WATER QUALITY TREATMENT BY LOW IMPACT TECHNIQUES

Low impact development (LID) techniques are beneficial by reducing stormwater runoff rates and volumes and by the inherent treatment provided by LID. LID should be encouraged in the study area for future developments and redevelopment/reconstruction projects.

## 12.9 PUBLIC EDUCATION

While not truly considered Post Construction treatment it is recommended that a public education campaign, be implemented in the study area. This would be an additional campaign to the overall City and County public education efforts.

Many locations were observed where neighboring property owners are using the adjacent channels, irrigation ditches, and ponds as dumping grounds. Piles of animal feces, waste construction materials, general trash and junk, yard clippings, tree trimmings, etc., were observed.

This public education campaign could also include developers and builders in the area to reinforce the need for proper Post Construction Controls.

# 13. MAJOR RECOMMENDATION SUMMARY, COST ESTIMATE AND PRIORITIZATION

## 13.1 SUMMARY OF RECOMMENDED IMPROVEMENTS AND COST ESTIMATE

An estimated opinion of probable construction cost has been prepared for each of the recommended improvements. The estimated cost is for the proposed stormwater improvement related items only. The estimated costs do not include costs for Engineering, property acquisition, easement acquisition, or related street or utility improvements/repairs.

It is noted that the cost estimates were prepared without the benefit of detailed surveys or engineering drawings. As such the estimated costs at final design and construction could vary significantly from those shown. The estimated costs are based on the professional judgement and experience of FMG Inc. FMG Inc., makes no warranty, either expressed or implied, that the cost of the work will not vary for these estimates.

TABLE 7
SUMMARY OF MAJOR IMPROVEMENTS AND COST ESTIMATE

| ELEMENT # | DESCRIPTION | ESTIMATED COST |
|---|---|---|
| 1 | Existing Pipe - No improvements. | NA |
| 2 | Existing Channel – Regrade upper segment as part of future subdivision. | SC |
| 3 | Existing Pipe - No improvements. | NA |
| 4 | Existing Channel – No improvements. | NA |
| 5 | Existing Pipe - No improvement. | NA |
| 6 | Existing Channel with Street Crossings – No improvements. | NA |
| 7 | Existing Channel with Street Crossings – No improvements. | NA |
| 8 | Existing Channel with Street Crossings – Stabilize upper reach of channel when development occurs. | NA |
| 9 | Existing Pipe - No improvement. | NA |
| 10 | Existing Channel with Street Crossings – No improvements. | NA |
| 11 | Existing Channel with Street Crossings – Add storm sewer and inlets on Plateau Lane to intercept and direct flows to Element 11 channel. | $120,000.00 |
| 12 | Existing Channel – No improvements. | NA |
| 13 | Existing Box Culvert – No improvements. | NA |
| 14 | Existing Channel and Street Crossing – Regrade channel and adjust drop structures. Improve Leroy Street box culvert inlet and reconstruct Leroy Street to create overflow section at box culvert | $155,000.00 |
| 15 | Existing box culvert – Improve culvert inlet and grade to create freeboard. | $62,000.00 |
| 16 | Existing Channel – No improvements, See Hawthorne Ditch for related improvement to leaking waste at upstream end of Element 16. | $5,000.00 |
| 17 | Existing Channel with Street crossing – Regrade channel and adjust drop structures. | $59,000.00 |
| 17A | Existing Channel with Street Crossing – Regrade channel and adjust drop structures. Improve overtopping section at Longview Road box culvert. | $111,000.00 |
| 18 | Existing Channel – Regrade channel and adjust drop structures. | $106,000.00 |
| 19 | Existing Channel – Regrade channel and adjust drop structures. | $108,000.00 |
| 20 | Existing Box Culvert – Adjust ditch block to create headwater. | $38,000.00 |
| 21 | Existing Channel – Improvements for water quality treatment only. | $19,000.00 |

TABLE 7 CONTINUED

| | | |
|---|---|---|
| 22 | Existing Channel with Street Crossing– Regrade channel and adjust drop structures. | $211,000.00 |
| 23 | Existing Storm Sewer – No Improvements. See Hawthorne Ditch for related improvement to leaking waste at upstream end of Element 16. | NA |
| 50 | Existing Storm Sewer – No improvements. | NA |
| 51 | Existing Small Storm Sewer – Replace with new channel. | $26,000.00 |
| 52 | Existing and New Channel – New channel north of Highway 44, No improvements to Highway 44 ditch, Improve driveway crossing Of Highway 44 ditch. | $68,000.00 |
| 53 | Existing Channel – No improvements. | NA |
| 100 | Existing Detention Pond – Regrade bottom and modify riser. | $94,000.00 |
| 101 | Existing Detention Pond – Add riser to outlet pipe and modify spillway. | $26,000.00 |
| 102 | Existing Detention Pond – Construct new outlet system and regrade top of dam. | $62,000.00 |
| 103 | Existing Detention Pond – Regrade bottom, raise dam, and modify riser. | $104,000.00 |
| 104 | Existing Detention Pond – Regrade bottom, add riser to outlet and modify spillway. | $42,000.00 |
| 105 | Existing Detention Pond – Regrade top of dam and construct new outlet weir. | $13,000.00 |
| 106 | New Detention Pond | $283,000.00 |
| 107 | New Detention Pond | $72,000.00 |
| Minor Basin 16 | On Site detention needed as part of all future developments. Verify and improve channel capacity as needed during future development. | NA |
| Minor Basin 17 | On Site detention needed as part of all future developments. Verify and improve channel capacity as needed during future development. | NA |
| Minor Basin 18 | On Site detention needed as part of all future developments. Verify and improve channel capacity as needed during future development. Prevent developed storm water from entering Little Giant Irrigation Ditch | NA |
| Hawthorne Ditch | Install new drainage system along Plateau Lane. Divert additional area to the Plateau Land drainage system. Modify irrigation ditch at Plateau Lane crossing with storm water overflow into storm sewer, siphon and enclosed pipe to east as required by the new Plateau Lane storm sewer. Grade ditch for new storm sewer to drain to Element 16. Repair waste gate and regrade overflow at existing siphon under Element 16. Replace pipe at Reed Court with box culvert. | $252,000.00 |
| Murphy Ditch | Repair leaking waste gate and regrade overflow at existing siphon under Element 18. Provide storm water diversion at Reservoir Road and near location where Element 52 will cross ditch. Design future development projects to prevent development storm water from entering Murphy Ditch. | $26,000.00 |
| Little Giant Ditch | Repair leaking waste gate at existing siphon under Element 22. Design future development projects to prevent development storm water from entering Little Giant Ditch. | $5,000.00 |
| | **TOTAL ESTIMATED COST** | **$2,067,000.00** |

*(SC: Subdivision Cost, Construction cost estimates have not been included for these tasks as the recommended improvements considered part of typical subdivision improvement costs.)*

## 13.2 PROJECT PRIORITIZATION

Following are proposed projects that should be given priority.

- Hawthorne Ditch Improvements

  It is recommended improvements to the Hawthorne Ditch be given the #1 priority in the study area. This includes all of the recommendation for the ditch, the proposed Plateau Lane storm sewer system, and the flow diversion on the west side of Plateau Lane to direct flow to the new storm sewer. These improvements are recommended as the highest priority because flooding problems are known to existing along the ditch, leaking gates/overflows are wasting water and contributing to the presence of wetland vegetation in downstream channels, and because Pennington County is proposing improvements to Plateau Lane.

- Element 15

  Improvements to Element 15 are judged to have the second priority because the current structure has less than 100 year capacity. The project is also very close to the Hawthorne Ditch improvements outlined as the #1 priority and could thus be combined as one project.

- Detention Pond 103

  This pond is judged to have third priority. Under existing conditions Pond 101 is on the verge of flowing over the spillway. As noted earlier the spillway channel is flat in cross section and overtopping flows may cause erosion at the edges. This erosion could potentially lead to failure of the dam.

  Improvements to Pond 103, under existing and future land use conditions, will significantly reduce flows to Pond 101. With existing land use conditions and with improved Pond 103, flows are reduced enough so about 3.5' of freeboard is available at Pond 101.

  Improvements to Pond 101 should not be completed before Pond 103 is improved. This is because any future improvements to Pond 101 are ultimately related to actual design of Pond 103.

- Detention Ponds 104 and 101

  These ponds are judged to have priority following improvements to Pond 103. The construction of these ponds, after construction of Pond 103, completes the full detention construction in this leg of the basin and flows to Element 15 will be approximately the same as the final DBDPA flows.

  If improvements to both ponds cannot be made at the same time it is recommended that Pond 104 be improved first.

- Element 17

  Element 17 is also included as a priority project because the improvements are not something that will be part of sub-division construction. The construction of the Hawthorne Ditch improvements will allow this project to be constructed without the problems caused by the base flow from the leaking gates. Adjacent homes also appear to be well above the channel and flooding from small events is judged unlikely even though floodwater may extend beyond the easement.

- Detention Ponds 102 and 105

  These ponds also need to be on the priority list because the lack of as-builts required assumptions for modeling. Under current pond conditions, including the upstream basin being nearly fully developed, the HMS model indicates the ponds will not overtop. However, due to the modeling uncertainties and assumptions that were required due to lack of as built data it is recommended these ponds be on the priority list to insure overtopping will not occur. At the very least, the ponds should be fully field investigated to determine the actual hydraulics when the remaining property in Sub-basins 14 and 15 is developed.

Improvements to other Elements as recommended can be completed as part of sub-division or development projects; as part of roadway improvement projects; or as funding become available for remaining projects. It is judged that any existing flooding that would occur at these Elements would (1) be of a nature that would not significantly damage structures although some damage may occur, (2) would be shallow, or (3) would result in shallow overtopping depths. It is also noted that many of these projects are best suited to design coordination with subdivision or development projects.

In the case of Detention Pond 106, it is necessary that the pond location and design be coordinated with the final subdivision layout and development density. Detention Pond 100 final design and improvements can then follow after Detention Pond 106.

# 14. MODELING COMPARISONS

## 14.1 DBDPA COMPARISON TO 1996 FERBER REPORT

This section compares the DBDPA peak discharges in this report to flows from the 1996 Ferber Engineering study as described in Section 2.1 of this report. As noted in Section 2.1 no documentation of City Council or County Commission approval of this 1996 report was found. Nevertheless, it is our understanding the 1996 Ferber study is the current document being used by City and County staff.

Contributing drainage areas to each site for each model are listed below the peak discharge.

The 1996 Ferber study is based on CUHP/UDSWM while the DBDPA uses the HMS model in this study. The results of this comparison are given in Table 8 below

TABLE 8
100 YEAR DBDPA FLOW COMPARISON TO 1996 FERBER REPORT

| LOCATION | 1996 FERBER REPORT Peak Discharge in CFS (Contributing Basin in Acres) | DBDPA Peak Discharges in CFS (Contributing Basin in Acres) |
|---|---|---|
| Discharge to Rapid Creek<br>*(Location is DBDP Junction 213)* | 916 cfs<br>(1036 Ac) | 1,230cfs<br>(1,440 Ac) |
| Highway 44<br>*(Location is DBDPA Junction 211)* | 969 cfs<br>(1036 Ac) | 990 cfs<br>(1132 Ac) |
| Albert Lane Box Culvert<br>*(Location is DBDPA Junction 207)* | 383 cfs<br>(710 Ac) | 503 cfs<br>(768 Ac) |
| Twilight Drive<br>*(Location is DBDPA Junction 206)* | 396 cfs<br>(710 Ac) | 445 cfs<br>(742 Ac) |

As indicated above there are significant differences in flows at various locations. These can be attributed to the different models but in this case it is also noted the differences in contributing basin size plays a major role.

In regards to the area at Rapid Creek, the Ferber study assumed the Longview Road area (sub-basins 12, 14, and 15 in this DBDPA) would flow across Highway 44 and then south to Rapid Creek rather than connect east to the main channel system as proposed in this study. If sub-basins 12, 14, and 15 were disconnected from the main channel the flow at Junction 213 would be reduced to 1,151 cfs (1,241 Acres) so the DBDPA still exceeds the Ferber report.

The 1996 Ferber study did not include any basin maps. However, a review of the 1996 Ferber data leads to the conclusion that the most downstream sub-basin inflow occurs where the main channel crosses the Murphy Ditch *(downstream end of Element 18 in DBDPA).* It is unknown why the remaining downstream area was not included in those calculations.

The basin sizes in this FMG DBDPA are believed to be more accurate than the 1996 study due to the availability of better maps and the ability to digitize the mapping information.

It is also noted that the Ferber study was based on Detention Pond 101 being at a different location than it exists. That study also assumed the area shown as Basin 8E in this DBDPA did not drain to the Albert Lane crossing. Refer to Section 3.9 for discussion regarding these issues. These two issues play a major role in the different flows at Albert Lane.

## 14.2 DBDPA SUB-BASINS CALCULATED WITH CUHP 2005

Another comparison was made by calculating peak discharges using the DBDPA sub-basin data with the current edition of CUHP. This current edition is CUHP 2005 and has enhancements to previous models including spreadsheet input. A notable change is that CUHP no longer uses Time of Concentration input data for basins less than 90 acres in size. Another change is that CUHP 2005 does not create input files for UDSWM because the Denver Urban Drainage District no longer used that routing model. Rather, they now use EPA SWMM for routing.

The results of this comparison are given in Table 9 below with discussion following the table.

TABLE 9
100 YEAR DBDPA SUB-BASIN PEAKS COMPARED TO CUHP METHODS

| SUB-BASIN NUMBER | CUHP 100 YEAR PEAK DISCHARGE (CFS) | HMS 100 YEAR PEAK DISCHARGE (CFS) | DUMMY CUHP 100 YEAR PEAK DISCHARGE WITH 2.65" ONE RAIN (CFS) |
|---|---|---|---|
| 1 | 782 | 612 | 680 |
| 2 | 283 | 246 | 247 |
| 3 | 251 | 170 | 219 |
| 4 | 379 | 280 | 328 |
| 5 | 689 | 472 | 601 |
| 6 | 450 | 282 | 395 |
| 7 | 178 | 163 | 154 |
| 8E | 74 | 70 | 64 |
| 8W | 197 | 97 | 171 |
| 9 | 230 | 165 | 200 |
| 10 | 196 | 136 | 169 |
| 11 | 320 | 261 | 278 |
| 12 | 238 | 212 | 211 |
| 13 | 174 | 196 | 150 |
| 14 | 249 | 165 | 216 |
| 15 | 108 | 95 | 94 |
| 16 | 459 | 284 | 403 |
| 17 | 129 | 72 | 111 |
| 18 | 182 | 134 | 158 |

The CUHP input assumed default values for various optional parameters which is the same as used on previous CUHP studies in Rapid City.

In all cases CUHP calculated higher peak flows than HMS. This is similar to what the City of Rapid City has discovered in other studies.

A significant reason for the higher CUHP flows is likely a result of CUHP converting the 2.95" rainfall input to a 3.41" modeling Hyetograph. This is about an 11% increase in rainfall volume over the 3.06" rainfall input into HMS.

A dummy CUHP run was made using 2.65" as the 1 hour rainfall input and the model then converted this to a 3.06" 2 hour rainfall to match the same rainfall as HMS. In this case the CUHP results compare more favorably with HMS but are still higher in most cases. Further reasonable explanation for higher CUHP values, even with reduced rain may be:

- CUHP default input data results in EIA being only slightly reduced from MIA. Input data for HMS uses MIA reduced to EIA using a Sutherland Equation. Those EIA values are lower than the EIA determined by the CUHP default.

- CUHP Snyder unit hydrograph calculations within the software utilize calibration data from the Denver Metropolitan area. It is unknown if those calibration values are actually applicable to Rapid City. It is known that FEMA will not allow these values to be used outside of the Denver area. Synder lag time methods in HMS were based on physical based input data which compared favorably to "typical" lag time coefficients in the RCIDCM.

- CUHP uses Horton's equation for infiltration losses. Initial and Final infiltration CUHP input is based on data for SCS Hydrologic Soil Groups from the Colorado area. It is unknown if these rates are applicable to the Rapid City area. By comparison, HMS uses the Green Ampt method and the input data are not considered "regional" as is the CUHP method, rather the input data is based on nationally recognized data.

- CUHP has an input for initial loss on impervious areas. HMS does not have this input option but rather accounts for this loss in the EIA.

No attempt was made to route the CUHP hydrographs through the basin. The purpose of this comparison is simply a brief comparison of the sub-basin flow predictions.

## 14.3 DBDPA COMPARISON TO NRCS CURVE NUMBER LOSS METHOD

A test of reasonableness of the HMS results was made by comparing the DBDPA model to results calculated by the NRCS Curve Number Methods. The NRCS method is nationally recognized. The NRCS method is the preferred or required method in many communities and states.

The NRCS calculations use the same data as the DBDPA model with the following exceptions.

- Curve Number infiltration method used in lieu of Green Ampt method.

- Curve Numbers are for pervious surfaces only rather than entering composite curve numbers that account for impervious surfaces.

- Percent imperviousness was entered as Mapped Impervious Area (MIA).

- Initial loss values for pervious surfaces were entered rather than using the NRCS default abstraction. The initial loss values are the same as the DBDPA data.

- Precipitation based on 24 hour Type 2 storm of 4.55 inches. A 24 hour storm is most commonly used with this model.

Results of this analysis are shown on Table 10 below.

TABLE 10
100 YEAR DBDPA FLOW COMPARISON TO NRCS CURVE NUMBER METHOD

| SUB-BASIN OR JUNCTION NUMBER | DBDPA 100 YEAR PEAK DISCHARGE (CFS) | NRCS CN METHOD 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|
| BASIN 1 | 612 | 400 |
| BASIN 2 | 246 | 160 |
| BASIN 3 | 170 | 125 |
| BASIN 4 | 280 | 194 |
| BASIN 5 | 472 | 339 |
| BASIN 6 | 282 | 211 |
| BASIN 7 | 163 | 108 |
| BASIN 8E | 70 | 46 |
| BASIN 8W | 97 | 73 |
| BASIN 9 | 165 | 109 |
| BASIN 10 | 136 | 91 |
| BASIN 11 | 261 | 179 |
| BASIN 12 | 212 | 157 |
| BASIN 13 | 196 | 145 |
| BASIN 14 | 165 | 117 |
| BASIN 15 | 95 | 65 |
| BASIN 16 | 284 | 207 |
| BASIN 17 | 72 | 52 |
| BASIN 18 | 134 | 86 |
| DP100 | 42 | 57 |
| DP101 | 101 | 99 |
| DP102 | 26 | 28 |
| DP103 | 42 | 42 |
| DP104 | 47 | 47 |
| DP105 | 36 | 37 |
| DP106 | 71 | 73 |
| DP107 | 80 | 73 |
| J201 | 247 | 171 |
| J202 | 209 | 165 |
| J203 | 282 | 225 |
| J204 | 164 | 115 |
| J205 | 288 | 254 |
| J206 | 445 | 359 |
| J207 | 503 | 390 |
| J208 | 557 | 446 |
| J209 | 672 | 505 |
| J210 | 790 | 592 |
| J211 | 991 | 718 |
| J212 | 1070 | 788 |
| J213 | 1231 | 905 |
| J250 | 95 | 66 |
| J251 | 212 | 158 |
| J260 | 284 | 207 |
| J261 | 72 | 52 |
| J262 | 134 | 86 |

The NRCS CN model predicts lower peak flows at all locations except at Detention Pond 100 where the flow is slightly higher. It is interesting to note that the Sub-basin peak flows in the DBDPA model fall between peak flows calculated by the CUHP and NRCS CN methods.

It is beyond the scope of the work to make an analysis of why the models predict different flows other than making the general statement that it is common to have different results from different models.

Tests indicated the use of the (1) City of Rapid City 2 hour storm rather than the 24 hour storm, (2) use EIA rather than MIA, and (3) use of the default NRCS Initial Abstraction; either by themselves or combined in any manner, would result in NRCS CN flows being less than shown.

## 14.4 HMS TEST WITH MODIFIED INPUT

A further test of the HMS model was made by using certain input data that is directly per the recommendations in the RCIDCM. The changed HMS input data for this comparison model is described as follows:

- Hydraulic conductivity is per RCIDCM Table 4-4. These values are ½ of what was used in the DBDPA input. These values would be hydraulic conductivity values that are cited in literature as being for bare soil. Reference Section 5.4.2. for discussion regarding the hydraulic conductivity values used in this DBDPA.

- MIA is reduced to EIA using the Average Sutherland Equation in the RCIDCM

Results of this analysis are shown on Table 11 below.

TABLE 11
100 YEAR DBDPA FLOW COMPARISON TO MODIFIED INPUT DATA

| SUB-BASIN OR JUNCTION NUMBER | DBDPA 100 YEAR PEAK DISCHARGE (CFS) | MODIFIED DATA 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|
| BASIN 1 | 612 | 663 |
| BASIN 2 | 246 | 259 |
| BASIN 3 | 170 | 182 |
| BASIN 4 | 280 | 307 |
| BASIN 5 | 472 | 501 |
| BASIN 6 | 282 | 295 |
| BASIN 7 | 163 | 177 |
| BASIN 8E | 70 | 75 |
| BASIN 8W | 97 | 108 |
| BASIN 9 | 165 | 178 |
| BASIN 10 | 136 | 146 |
| BASIN 11 | 261 | 273 |
| BASIN 12 | 212 | 218 |
| BASIN 13 | 196 | 207 |
| BASIN 14 | 165 | 183 |
| BASIN 15 | 95 | 102 |
| BASIN 16 | 284 | 299 |
| BASIN 17 | 72 | 74 |
| BASIN 18 | 134 | 140 |
| DP100 | 42 | 46 |

TABLE 11 CONTINUED
100 YEAR DBDPA FLOW COMPARISON TO ADJUSTED INPUT DATA

| SUB-BASIN OR JUNCTION NUMBER | DBDPA 100 YEAR PEAK DISCHARGE (CFS) | MODIFIED DATA 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|
| DP101 | 101 | 105 |
| DP102 | 26 | 29 |
| DP103 | 42 | 47 |
| DP104 | 47 | 49 |
| DP105 | 36 | 42 |
| DP106 | 71 | 75 |
| DP107 | 80 | 83 |
| J201 | 247 | 260 |
| J202 | 209 | 222 |
| J203 | 282 | 296 |
| J204 | 164 | 178 |
| J205 | 288 | 324 |
| J206 | 445 | 487 |
| J207 | 503 | 551 |
| J208 | 557 | 611 |
| J209 | 672 | 742 |
| J210 | 790 | 870 |
| J211 | 991 | 1083 |
| J212 | 1070 | 1164 |
| J213 | 1231 | 1339 |
| J250 | 95 | 103 |
| J251 | 212 | 218 |
| J260 | 284 | 299 |
| J261 | 72 | 74 |
| J262 | 134 | 140 |

The results of the HMS model modified to use RCIDCM recommended data for hydraulic conductivity and EIA reduction results in increased flows at all locations. This leads to the conclusion that the model is sensitive to the hydraulic conductivity input and that particular parameter appears to have more weight in the results than the reduced imperviousness values created by the Average Sutherland equation. Because the model appears sensitive to the hydraulic conductivity value it is important that the earlier recommendations for appropriate topsoil, topsoil amendments, loosening of upper soils etc., be required in future developments.

This run also illustrates the importance to have freeboard to maintain orifice flow at the metering dams.

It is noted this test is only for comparison purposes. The results of the DBDPA HMS models have been judged to be reasonable and are believed to be based on data that is more appropriate than the "bare" soil hydraulic conductivity and the Average Sutherland Equation used in the comparison model. Even if there is some urban reduction of vegetated hydraulic conductivity, and assuming the values in published literature are reasonable, it seems reasonable that existing vegetation and future vegetation/grading requirements will result in the hydraulic conductivity to a level that is better than the bare soil conditions.

## 14.5 HMS MODEL WITH FUTURE LAND USE AND EXISTING DETENTION

This model is informational to illustrate flows that would result assuming the study area is allowed to develop to the future land use conditions and no changes are made to the existing detention pond system. Proposed ponds 106 and 107 are not included in this run. The remaining routing elements are the same as the DBDPA conditions.

Results of this analysis are shown on Table 12 below.

TABLE 12
100 YEAR DBDPA COMPARED TO FUTURE LAND USE AND EXISTING DETENTION

| SUB-BASIN OR JUNCTION NUMBER | DBDPA 100 YEAR PEAK DISCHARGE (CFS) | MODIFIED DATA 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|
| BASIN 1 | 612 | 612 |
| BASIN 2 | 246 | 246 |
| BASIN 3 | 170 | 170 |
| BASIN 4 | 280 | 280 |
| BASIN 5 | 472 | 472 |
| BASIN 6 | 282 | 282 |
| BASIN 7 | 163 | 163 |
| BASIN 8E | 70 | 70 |
| BASIN 8W | 97 | 97 |
| BASIN 9 | 165 | 165 |
| BASIN 10 | 136 | 136 |
| BASIN 11 | 261 | 261 |
| BASIN 12 | 212 | 212 |
| BASIN 13 | 196 | 196 |
| BASIN 14 | 165 | 165 |
| BASIN 15 | 95 | 95 |
| BASIN 16 | 284 | 284 |
| BASIN 17 | 72 | 72 |
| BASIN 18 | 134 | 134 |
| DP100 | 42 | 92 |
| DP101 | 101 | 120 |
| DP102 | 26 | 2 |
| DP103 | 42 | 84 |
| DP104 | 47 | 72 |
| DP105 | 36 | 37 |
| J201 | 247 | 259 |
| J202 | 209 | 229 |
| J203 | 282 | 716 |
| J204 | 164 | 175 |
| J205 | 288 | 328 |
| J206 | 445 | 491 |
| J207 | 503 | 550 |
| J208 | 557 | 604 |
| J209 | 672 | 720 |
| J210 | 790 | 840 |
| J211 | 991 | 1036 |
| J212 | 1070 | 1136 |

TABLE 12 CONTINUED
100 YEAR FLOWS COMPARED TO FUTURE LAND USE AND EXISTING DETENTION

| SUB-BASIN OR JUNCTION NUMBER | DBDPA 100 YEAR PEAK DISCHARGE (CFS) | MODIFIED DATA 100 YEAR PEAK DISCHARGE (CFS) |
|---|---|---|
| J213 | 1231 | 1306 |
| J250 | 95 | 95 |
| J251 | 212 | 212 |
| J260 | 284 | 284 |
| J261 | 72 | 72 |
| J262 | 134 | 134 |

The results of this run justify the need for the improved and added detention ponds.

Under this scenario the flow at Junction 207 (Element 15 box culvert) is about 50 cfs higher than the DBPDA flow. This 550 cfs is high enough that the proposed inlet improvements would not be a feasible solution and a larger box, additional pipe, or property purchase would have been necessary. None of those options was judged acceptable.

The flow in Element 8 and at Junction 203 is high. This would have resulted in Pond 100 filling to such a depth that pond modifications to help at Element 15 would not be reasonable. As such Pond 106 is proposed in the DBDPA. The flows in the channel are also high and, while not investigated, may cause stability issues in the channel.

Under this scenario Pond 101 will discharge over the spillway. It was earlier noted the existing spillway has issues related to edge protection.

Under this scenario Pond 103 is within 6" of spilling into the existing riser. Spill into the riser would have downstream consequences related to higher flows.

Under this scenario Pond 104 filled right to the spillway elevation and would likely spill if there were any clogging of the outlet pipe.

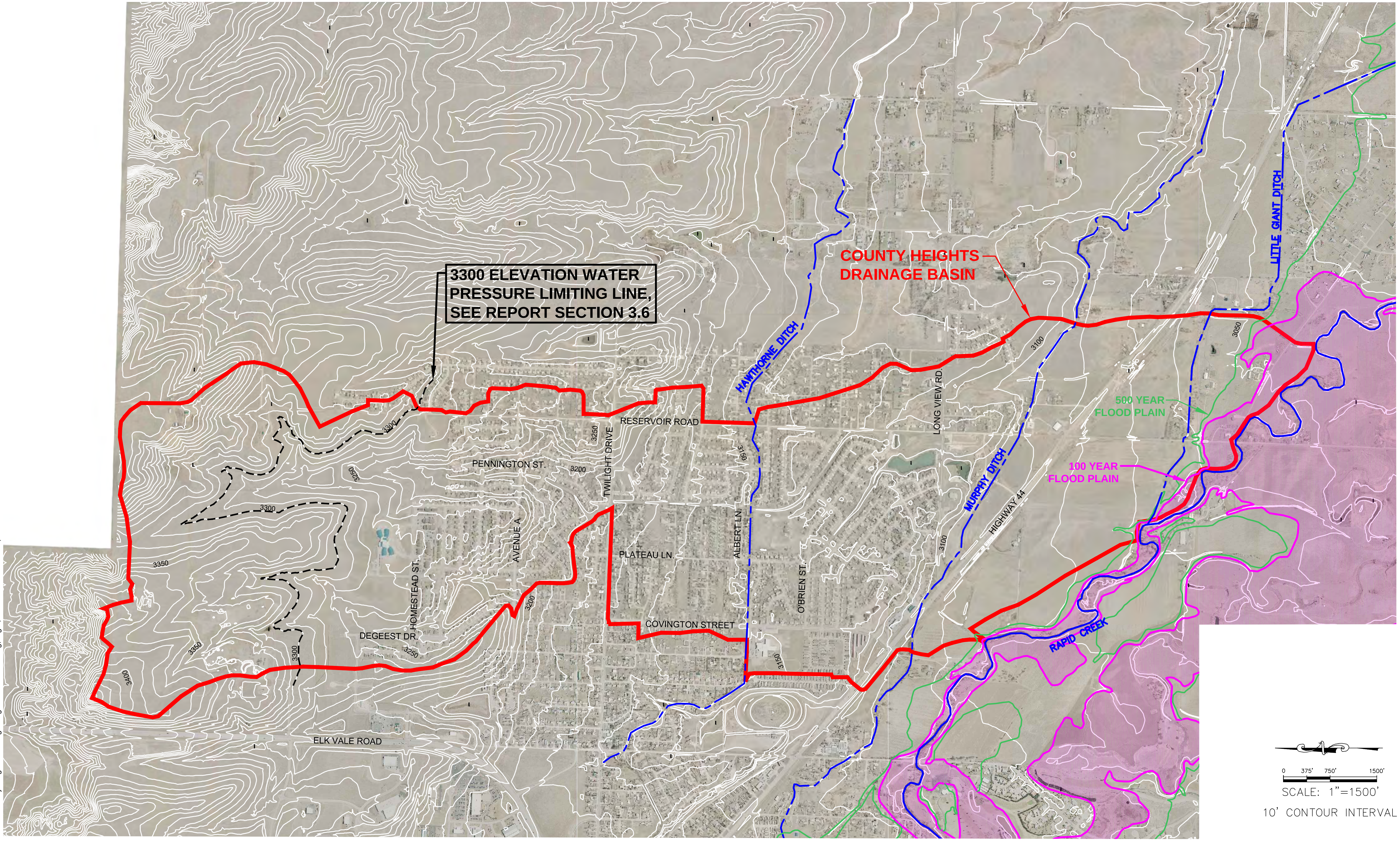
Among other things the Reservoir Road box culvert will spill, simple berming for capacity improvements may not be possible at the Highway 44 box culvert, and Highway 44 and Reservoir Road will both overtop at Junction 251.

## 14.6 REASONABLENESS OF RESULTS STATEMENT

Based on review and comparison of the DBDPA HMS results, input data, assumptions, engineering judgement, etc.; and upon comparison to other models per discussion in the above sections, it is judged the HMS modeling results presented in the report are reasonable.

(END OF REPORT NARRATIVE)

FLOODPLAIN BOUNDARIES OBTAINED FROM FEMA PANEL 792, DATED JUNE 2013.

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Public Works Department
Engineering Division

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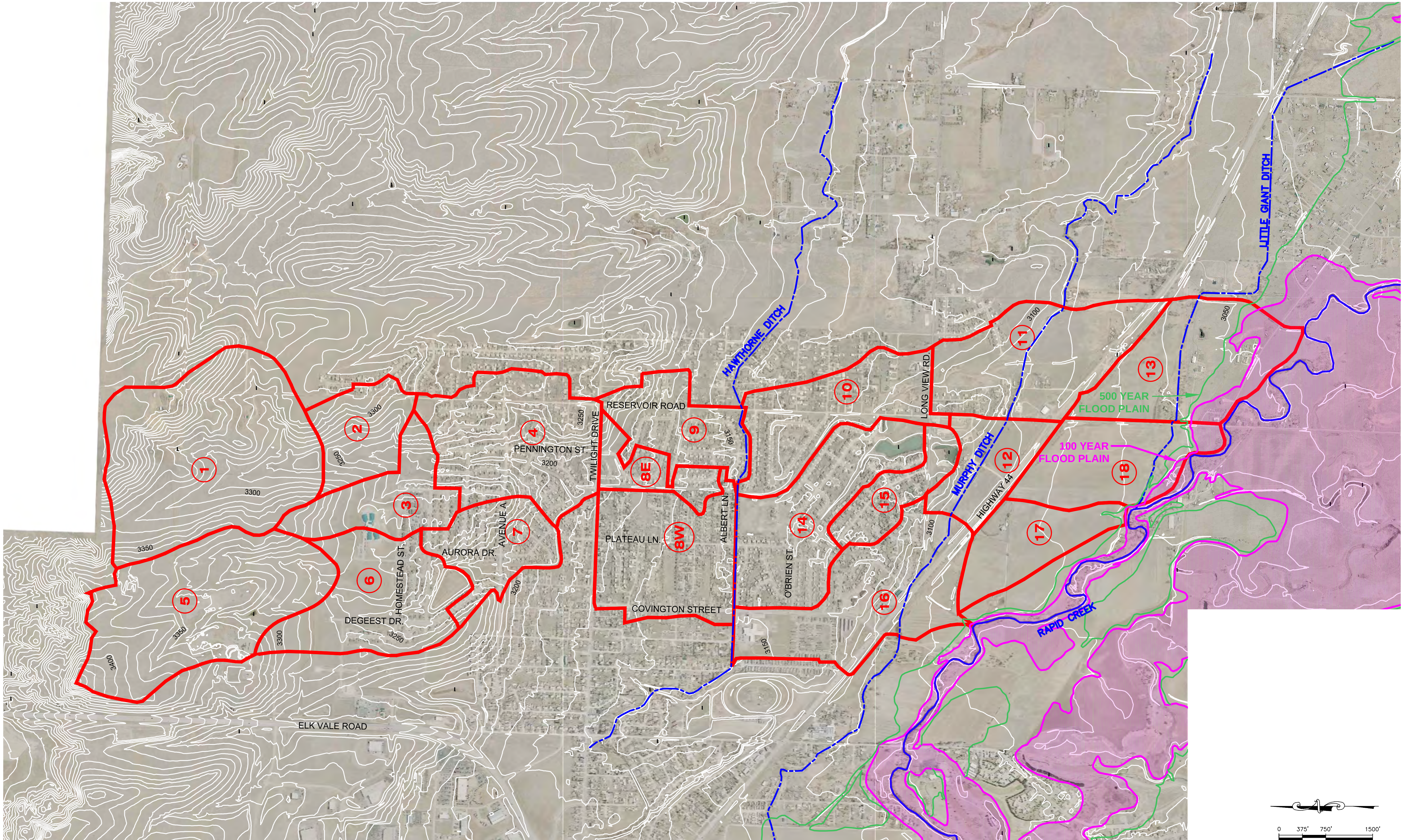
COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
STUDY AREA WITH FEMA FLOODPLAIN BOUNDARIES

Sheet:
FIGURE 1

H:\110136 County Heights Drainage\dwg\110136 Flood.dwg, Fig 1, 1/20/2016 7:38:51 AM, 1:1




SCALE: 1"=1500'

10' CONTOUR INTERVAL

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Revisions:

COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: SUB BASIN MAP

Sheet: **FIGURE 2**

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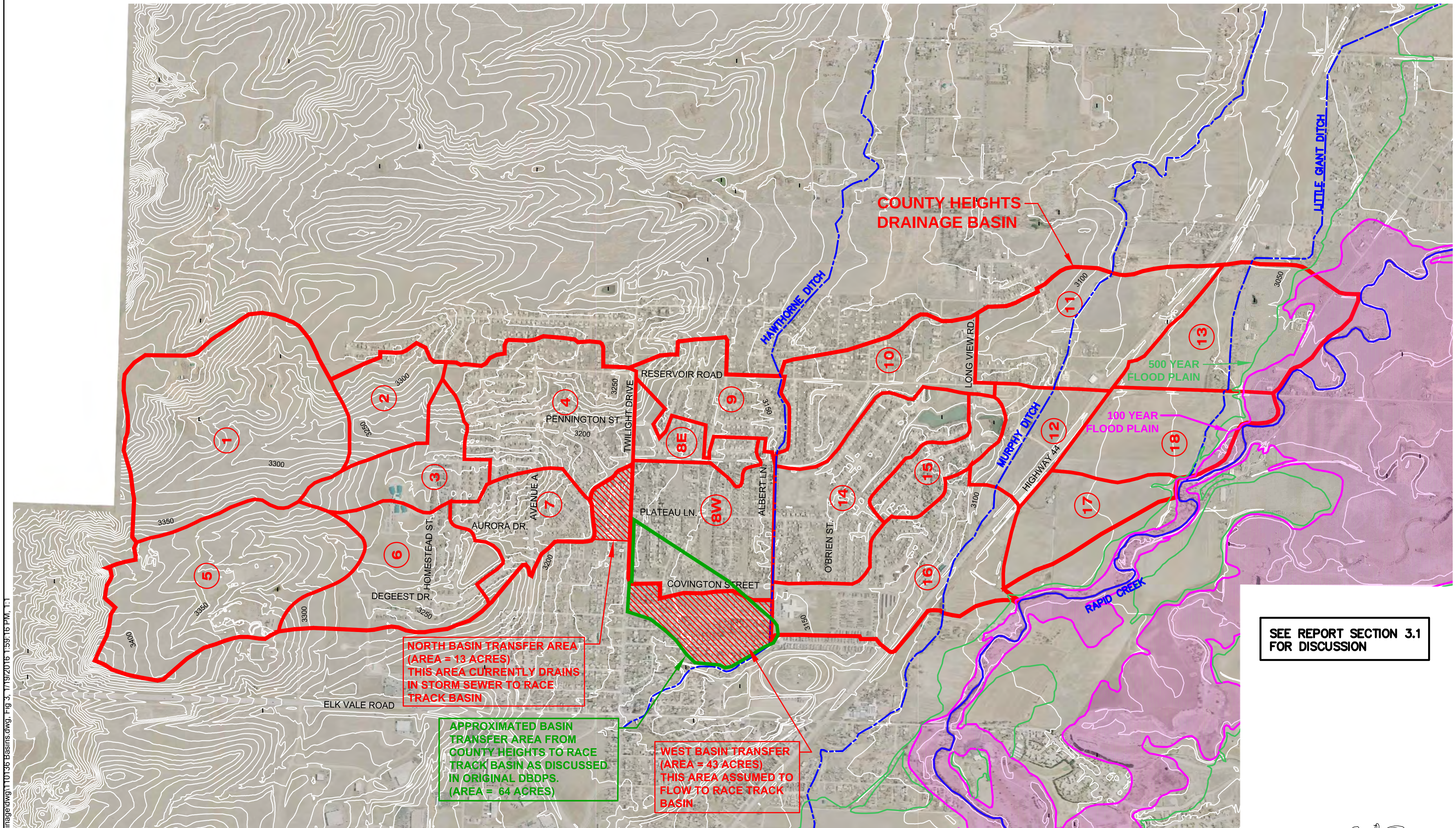


0 375' 750' 1500'

SCALE: 1"=1500'

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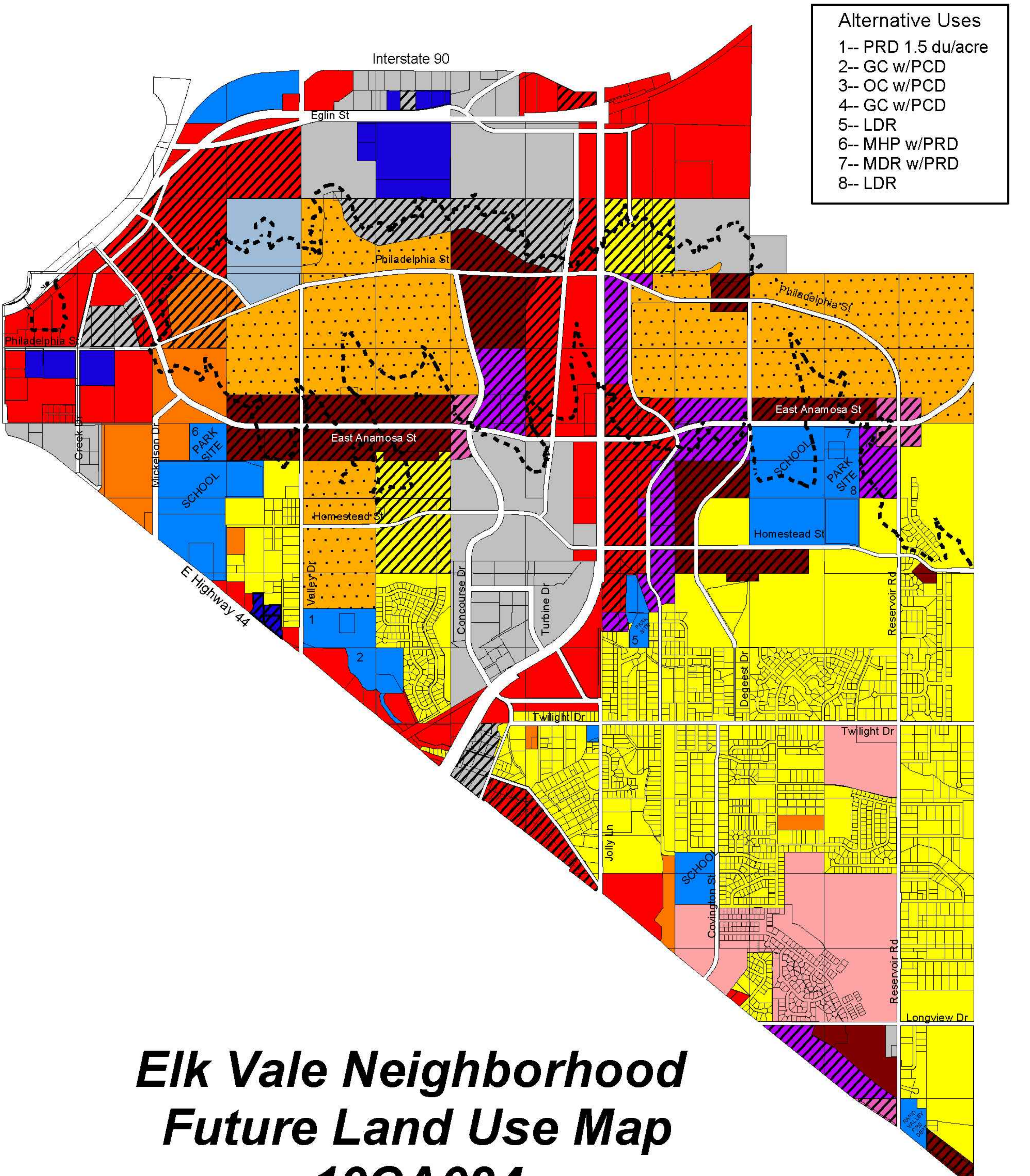
COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
BASIN
TRANSFER
MAP

Sheet:
**FIGURE 3**




# Elk Vale Neighborhood Future Land Use Map
# 10CA034
# Scale 1" = 1800'
# 10/04/2010

**FIGURE 4**

## Future Land Use Legend

| | | |
|---|---|---|
| LDR | PRD 1.5 du/ac | LI |
| LDR w/PRD | PUD | LI w/PLID |
| MDR | NC w/PCD | HI |
| MDR w/PRD | OC w/PCD | HI w/PID |
| MHP | GC | Public |
| MHP w/PRD | GC w/PCD | ROW |
| | BP | Drainage Area |

Above Elevation 3300' - Special Improvements For Adequate Services Are Necessary

Prepared By
Rapid City Area
Metropolitan
Planning
Organization

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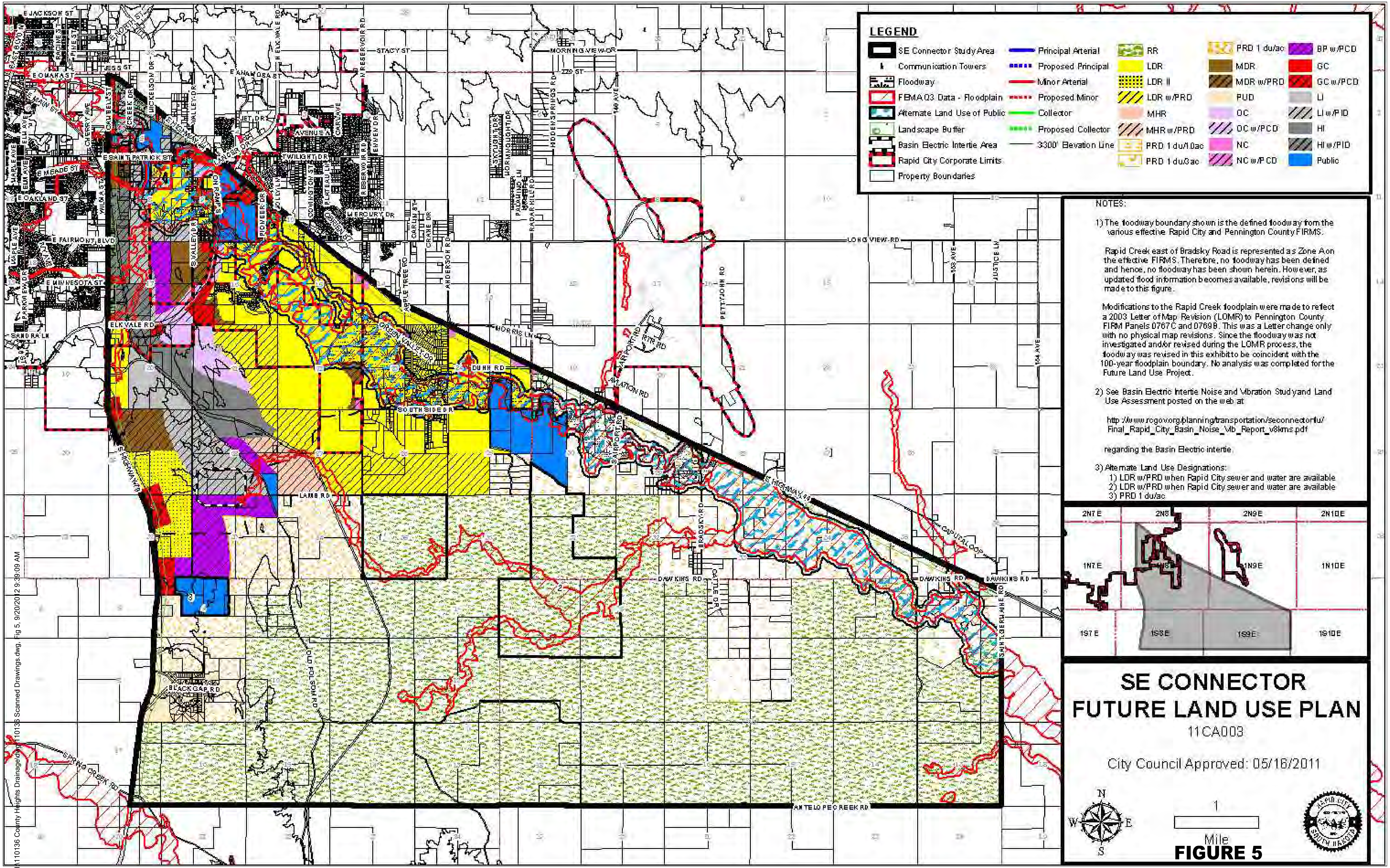
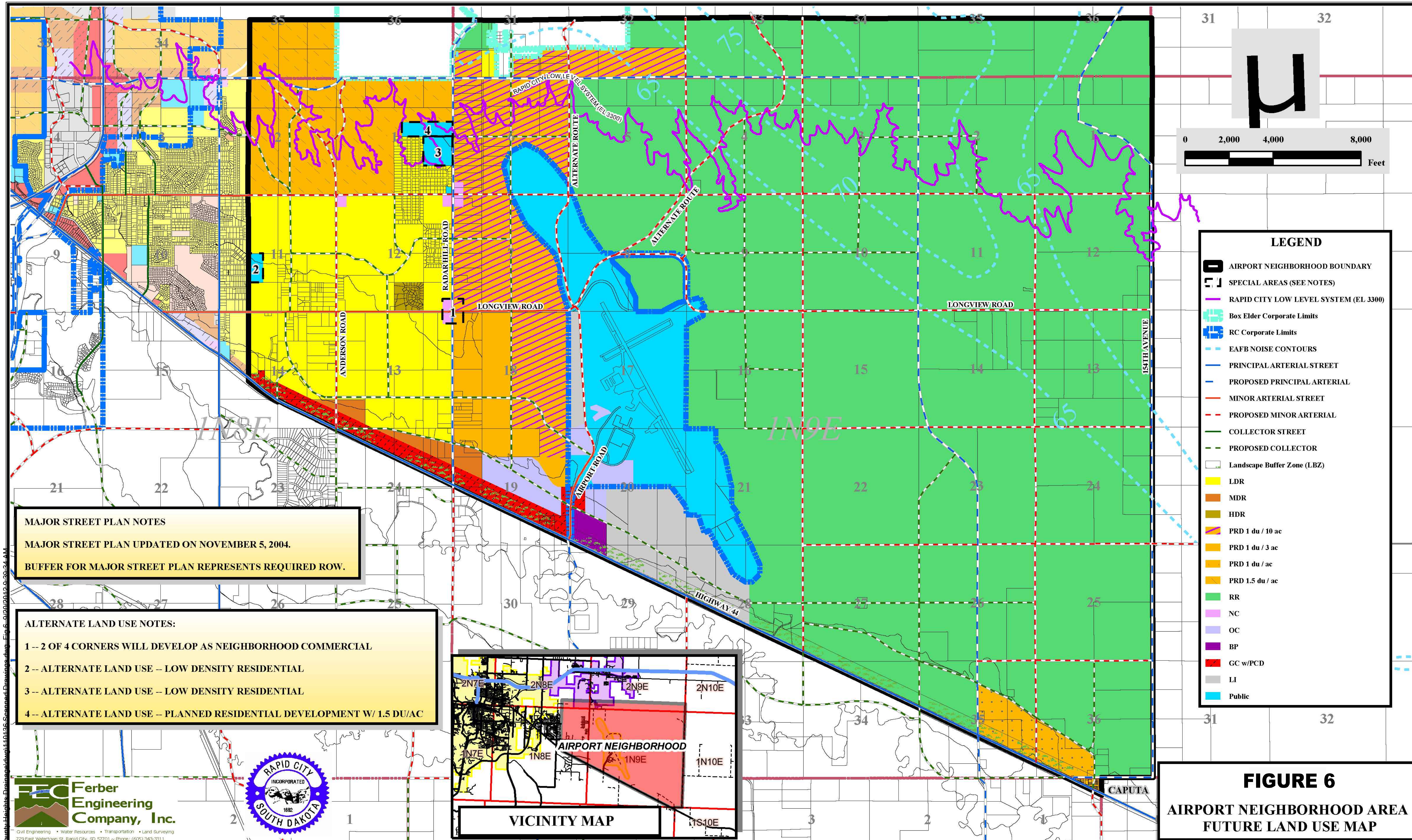
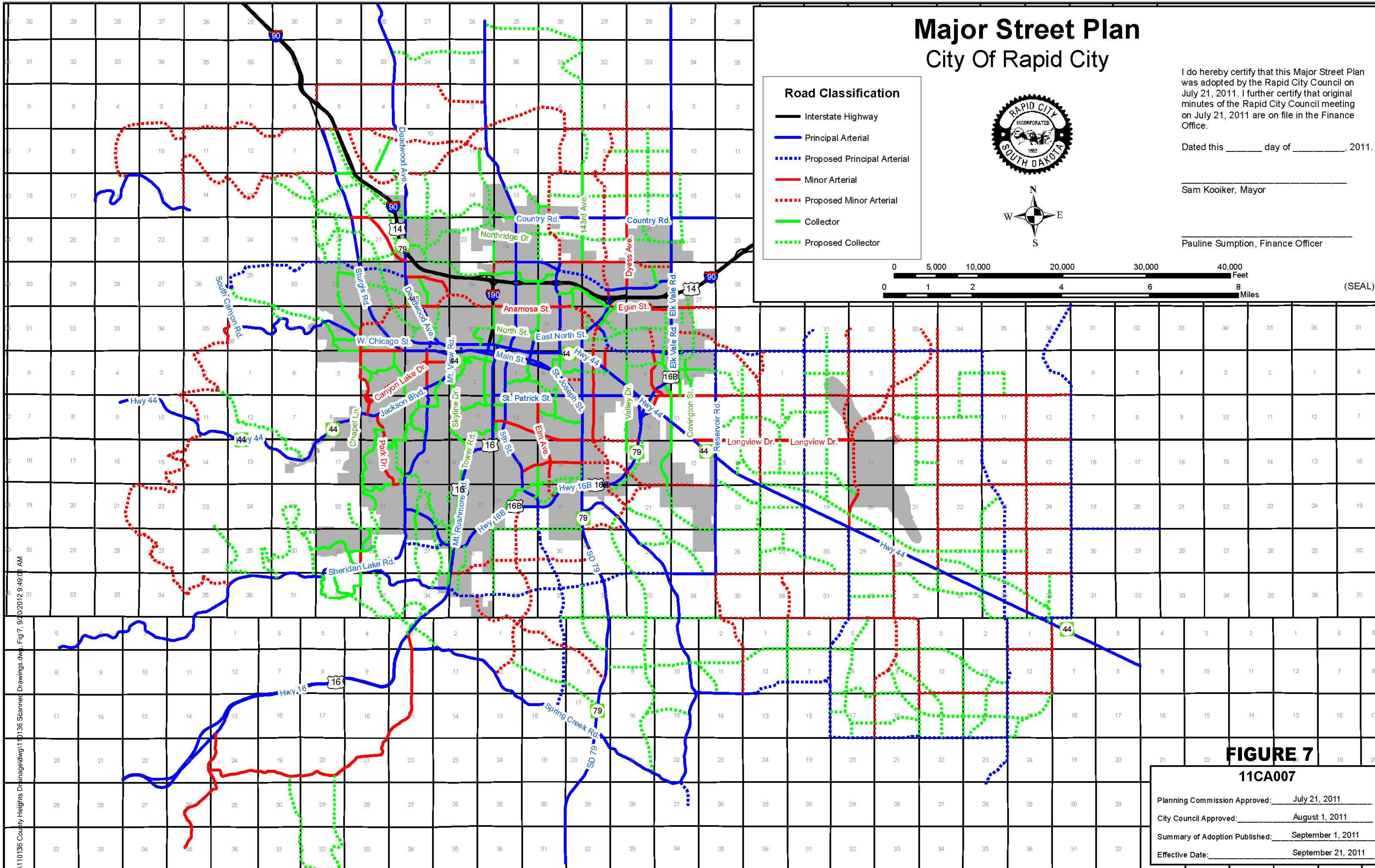
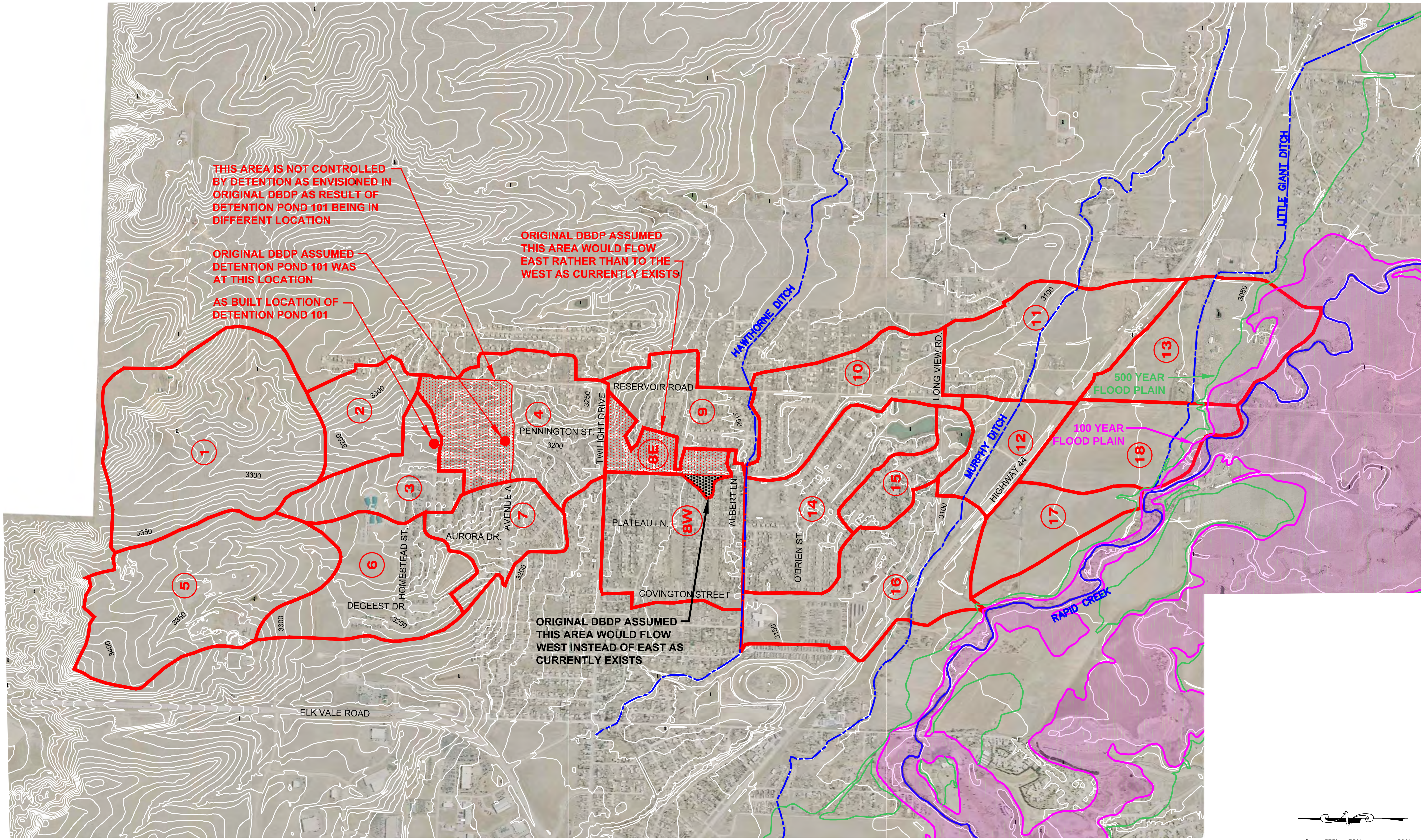



H:\110136 County Heights Drainage\110136 Scanned Drawings.dwg, Fig 5, 9/20/2012 9:39:09 AM


MAJOR STREET PLAN NOTES
MAJOR STREET PLAN UPDATED ON NOVEMBER 5, 2004.
BUFFER FOR MAJOR STREET PLAN REPRESENTS REQUIRED ROW.
ALTERNATE LAND USE NOTES:
1 -- 2 OF 4 CORNERS WILL DEVELOP AS NEIGHBORHOOD COMMERCIAL
2 -- ALTERNATE LAND USE -- LOW DENSITY RESIDENTIAL
3 -- ALTERNATE LAND USE -- LOW DENSITY RESIDENTIAL
4 -- ALTERNATE LAND USE -- PLANNED RESIDENTIAL DEVELOPMENT W/ 1.5 DU/AC
LEGEND
AIRPORT NEIGHBORHOOD BOUNDARY
SPECIAL AREAS (SEE NOTES)
RAPID CITY LOW LEVEL SYSTEM (EL 3300)
Box Elder Corporate Limits
RC Corporate Limits
EAFB NOISE CONTOURS
PRINCIPAL ARTERIAL STREET
PROPOSED PRINCIPAL ARTERIAL
MINOR ARTERIAL STREET
PROPOSED MINOR ARTERIAL
COLLECTOR STREET
PROPOSED COLLECTOR
Landscape Buffer Zone (LBZ)
LDR
MDR
HDR
PRD 1 du / 10 ac
PRD 1 du / 3 ac
PRD 1 du / ac
PRD 1.5 du / ac
RR
NC
OC
BP
GC w/PCD
LI
Public
0 2,000 4,000 8,000
Feet
LONGVIEW ROAD
ANDERSON ROAD
RADAR HILL ROAD
ALTERNATE ROUTE
AIRPORT ROAD
154TH AVENUE
HIGHWAY 44
RAPID CITY LOW LEVEL SYSTEM (EL 3300)
1N8E
1N9E
CAPUTA
VICINITY MAP
AIRPORT NEIGHBORHOOD
2N7E
2N8E
2N9E
2N10E
1N7E
1N8E
1N9E
1N10E
1S10E
RAPID CITY
INCORPORATED
1882
SOUTH DAKOTA
Ferber Engineering Company, Inc.
FIGURE 6
AIRPORT NEIGHBORHOOD AREA
FUTURE LAND USE MAP


Major Street Plan
City Of Rapid City
Road Classification
Interstate Highway
Principal Arterial
Proposed Principal Arterial
Minor Arterial
Proposed Minor Arterial
Collector
Proposed Collector
RAPID CITY INCORPORATED 1882 SOUTH DAKOTA
N
W
E
S
0 5,000 10,000 20,000 30,000 40,000 Feet
0 1 2 4 6 8 Miles
I do hereby certify that this Major Street Plan was adopted by the Rapid City Council on July 21, 2011. I further certify that original minutes of the Rapid City Council meeting on July 21, 2011 are on file in the Finance Office.
Dated this _______ day of __________, 2011.
Sam Kooiker, Mayor
Pauline Sumption, Finance Officer
(SEAL)
FIGURE 7
11CA007
Planning Commission Approved: July 21, 2011
City Council Approved: August 1, 2011
Summary of Adoption Published: September 1, 2011
Effective Date: September 21, 2011
H:\110136 County Heights Drainage\dwg\110136 Scanned Drawings.dwg, Fig 7, 9/20/2012 9:49:01 AM

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COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
SUB BASIN 8E FLOW DIRECTION & DETENTION POND 101 LOCATION MAP

Sheet:
**FIGURE 8**

H:\110136 County Heights Drainage\dwg\110136 Basins.dwg, Fig 8, 1/19/2016 10:25:03 AM, 1:1

# POTENTIAL WETLAND KEY NOTES

(KEY NOTES USE DIFFERENT REFERENCE NUMBERS THAN HYDRAULIC ROUTING)

1. Water body present on NRCS Aerial Photos.
2. Natural grassy channel located downstream of water body. No base flow.
3. Natural grassy channel identified as blue intermittent stream line on USGS quad. No base flow observed.
4. Graded grass lined channel with spot areas of hydrophytic vegetation observed. Majority of channel being mowed. No base flow observed.
5. Hydrophytic vegetation observed over most of detention pond bottom. No base flow observed.
6. Graded grass lined channel. Mowed through most of length. No base flow observed.
7. Graded grass lined channel. Mowed through most of length. No base flow observed.
8. Graded channel. Hydrophytic vegetation observed generally full length and width of channel. No base flow observed.
9. Hydrophytic vegetation observed over most of detention pond bottom. No base flow observed.
10. Hydrophytic vegetation observed over most of detention pond bottom. No base flow observed.
11. Natural and graded grass lined channel. No base flow observed. Headcutting upstream of detention pond slope observed. Identified as blue intermittent stream line on USGS quad.
12. Bottom of detention pond. Minimal to no hydrophytic vegetation observed. Highlighted because of nearby NWI identification and because blue intermittent stream line on USGS quad goes through pond. No base flow observed.
13. Natural grassy channel identified as blue intermittent stream line on USGS quad. No base flow observed.
14. Graded grass and weed lined channel with gabion drop structures. No base flow observed. Mowed and un-mowed segments.
15. Graded grass and weed lined channel with gabion drop structures. No base flow observed. Moved and unmowed segments.
16. Graded channel. Trickle base flow observed at upstream end. Base flow observed at downstream end and entering channel from Reservoir Road 54" Storm Sewer. Hydrophytic vegetation observed generally full length and width of channel.
17. Graded channel with gabion drop structures. Base flow observed. Hydrophytic vegetation observed generally full length and width of channel.
18. Graded channel with gabion drop structures. Base flow observed. Hydrophytic vegetation observed generally full length and width of channel.
19. Area of hydrophytic vegetation that is adjacent to highway.
20. Graded channel with gabion drop structures. Base flow observed. Hydrophytic vegetation observed generally full length and width of channel.
21. Graded grass lined channel with spot areas of hydrophytic vegetation observed. Most of channel being mowed. No base flow observed.
22. Graded channel. Trickle flow observed. Hydrophytic vegetation, including significant growth of cottonwood saplings and woody shrubs, observed generally full length and width of channel.
23. Water body with hydrophytic vegetation along shoreline.
24. Water body with hydrophytic vegetation along shoreline.
25. Area of hydrophytic vegetation that is adjacent to abandoned railroad.
26. Historic manmade drainage paths with areas of hydrophytic vegetation observed.

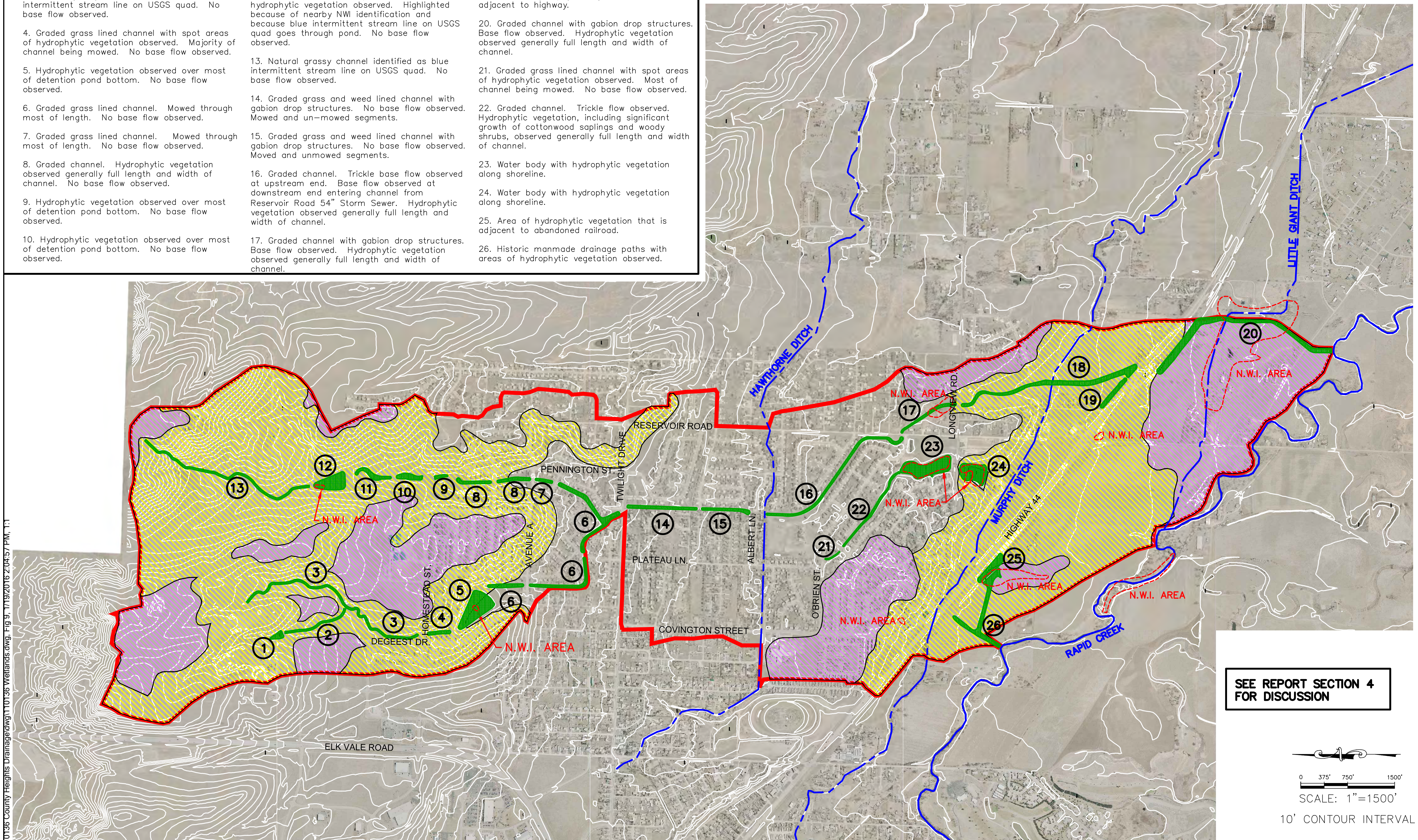
## SOILS LEGEND

- HYDRIC SOILS
- SOIL WITH HYDRIC INCLUSIONS
- NON HYDRIC SOILS

## WETLANDS LEGEND

- WETLANDS SHOWN ON USFWS WETLAND INVENTORY MAP
- POTENTIAL WETLANDS

NOTE: OTHER ISOLATED AREAS OF POTENTIAL WETLANDS MAY EXIST




**SEE REPORT SECTION 4 FOR DISCUSSION**

0 375' 750' 1500'

SCALE: 1"=1500'

10' CONTOUR INTERVAL

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| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: POTENTIAL WETLANDS

Sheet: FIGURE 9

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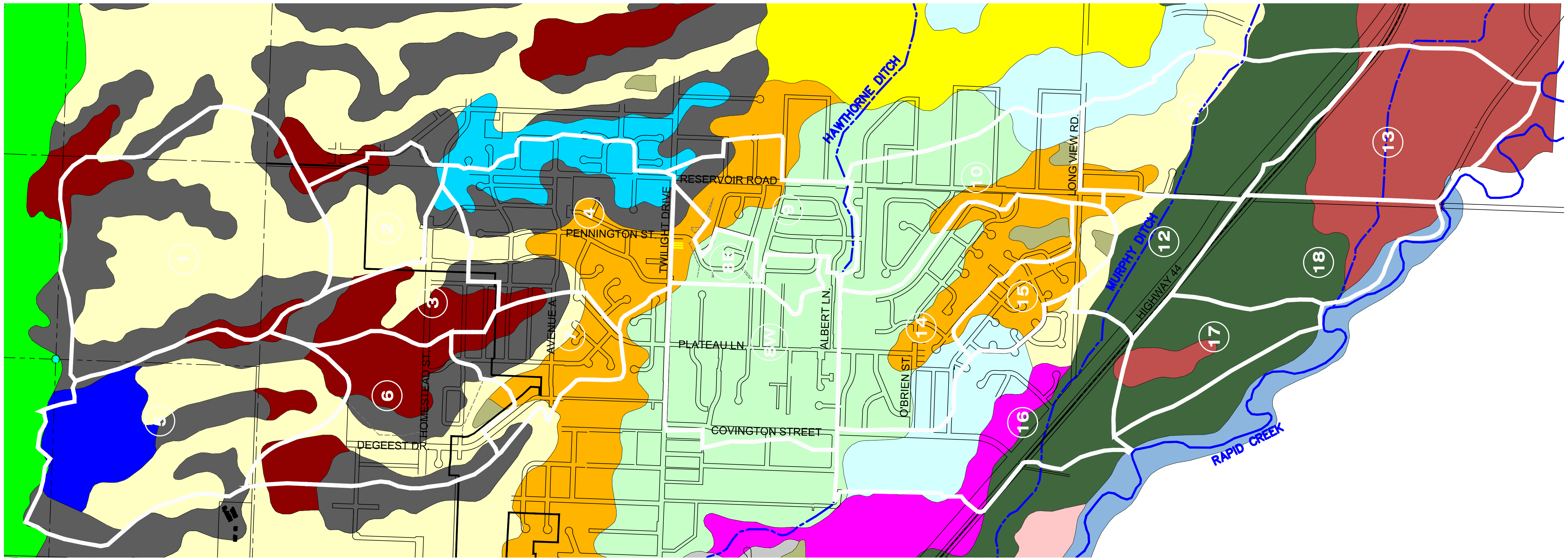


SOIL TYPES

- Cn
- Co
- Eg
- EnD
- NdE
- NuB
- NuC
- NwA
- Ow
- ScE
- SeA
- SeB
- SkA
- SkB
- SmC
- W
- ZnD

SEE REPORT SECTION 5 FOR DISCUSSION

0 375' 750' 1500'

SCALE: 1"=1500'

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| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: SOILS MAP

Sheet: FIGURE 10

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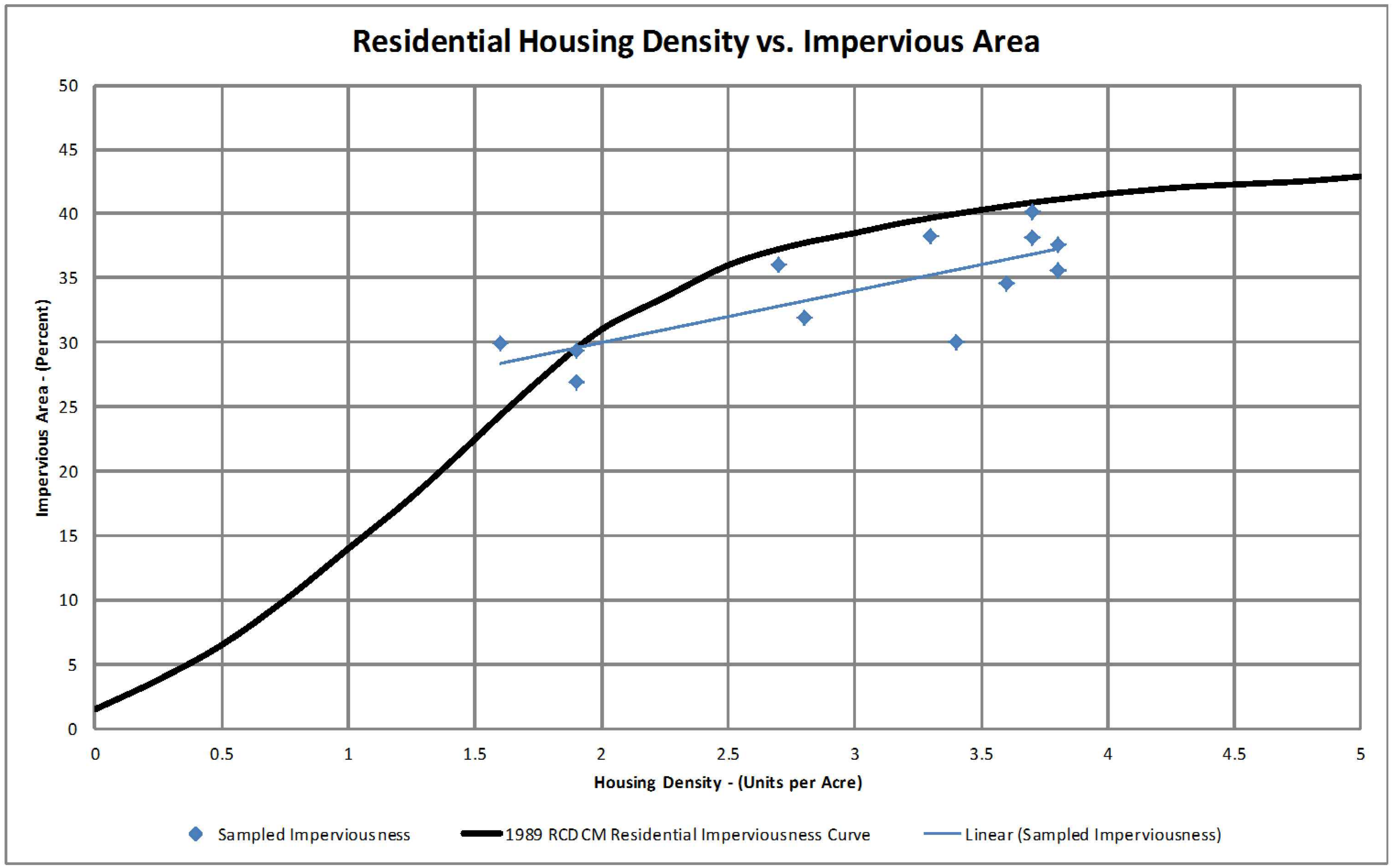


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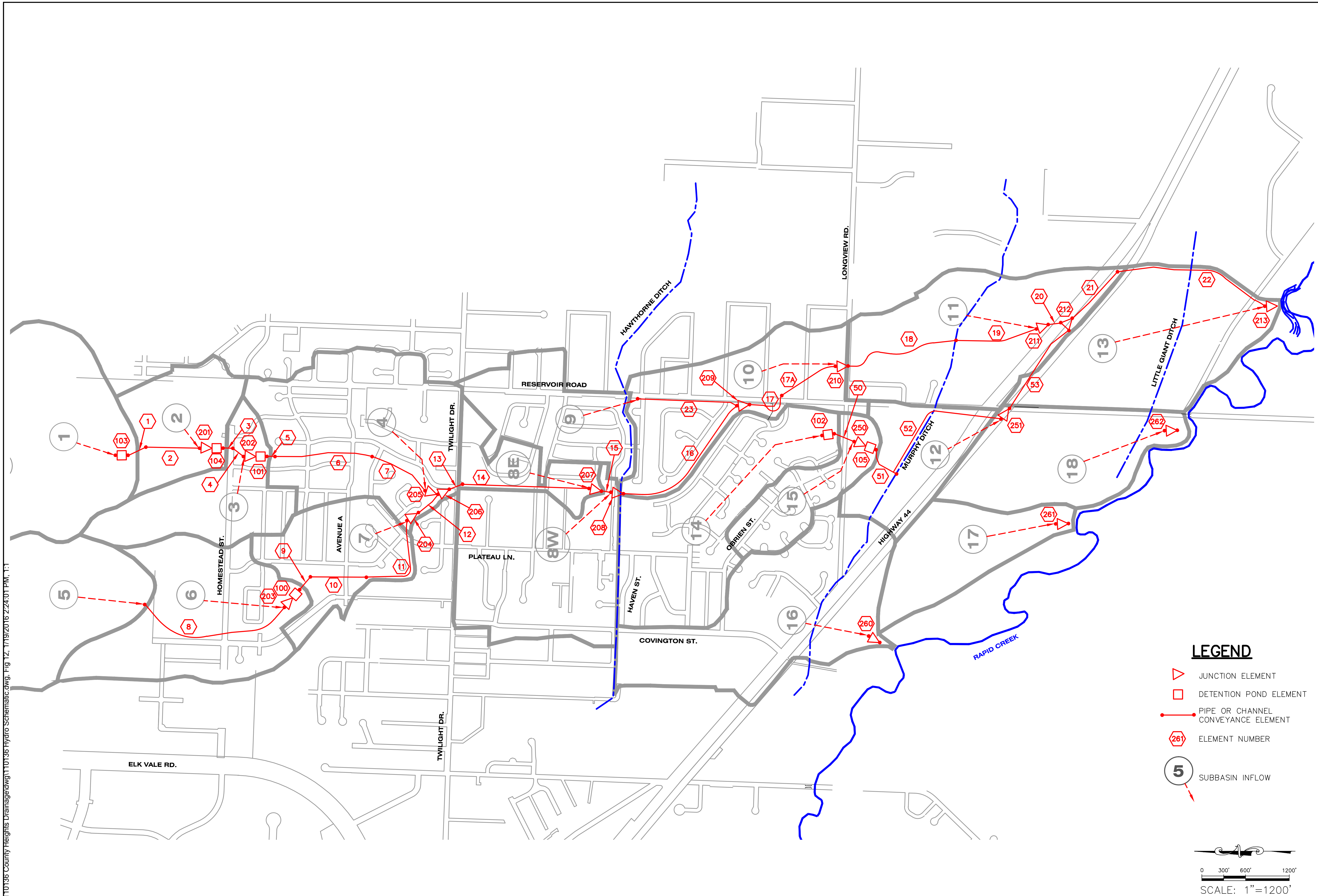
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
IMPERVIOUS COMPARISON CHART

Sheet:
**FIGURE 11**




H:\110136 County Heights Drainage\dwg\110136 Hydro Schematic.dwg, Fig 12, 1/19/2016 2:24:01 PM, 1:1

Prepared For: Public Works Department

Engineering Division

Prepared By:

**F M G, Inc.**

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Rapid City, SD 57702-0317
(605) 342-4105 FAX (605) 342-4222
www.fmgengineering.com

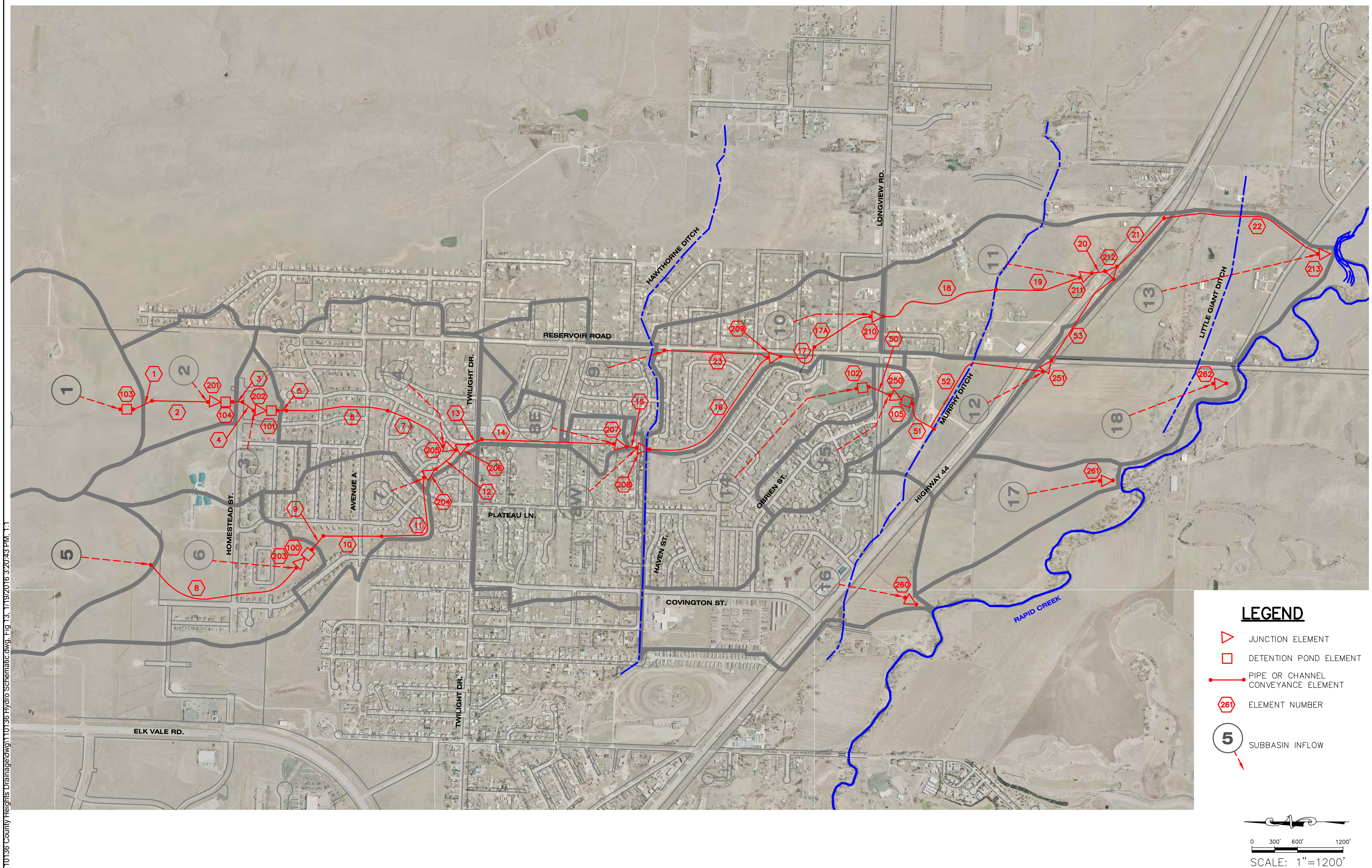
| | |
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: EXISTING HYDROLOGIC ROUTING NETWORK

Sheet: **FIGURE 12**




H:\110136 County Heights Drainage\dwg\110136 Hydro Schematic.dwg, Fig 13, 1/19/2016 3:20:43 PM, 1:1

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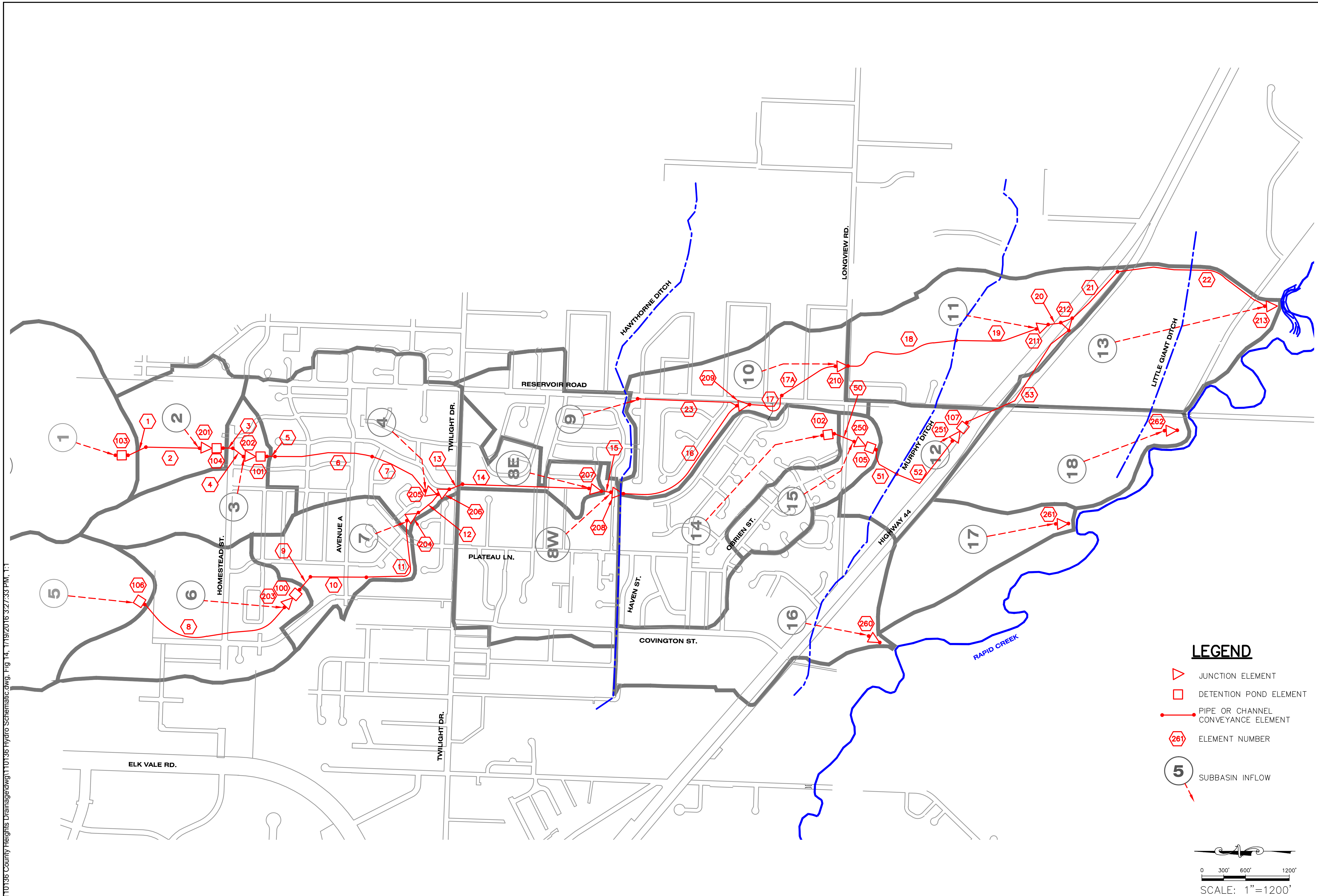
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: EXISTING HYDROLOGIC ROUTING NETWORK WITH AERIAL

Sheet: FIGURE 13




H:\110136 County Heights Drainage\dwg\110136 Hydro Schematic.dwg, Fig 14, 1/19/2016 3:27:33 PM, 1:1

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Engineering Division

Prepared By:
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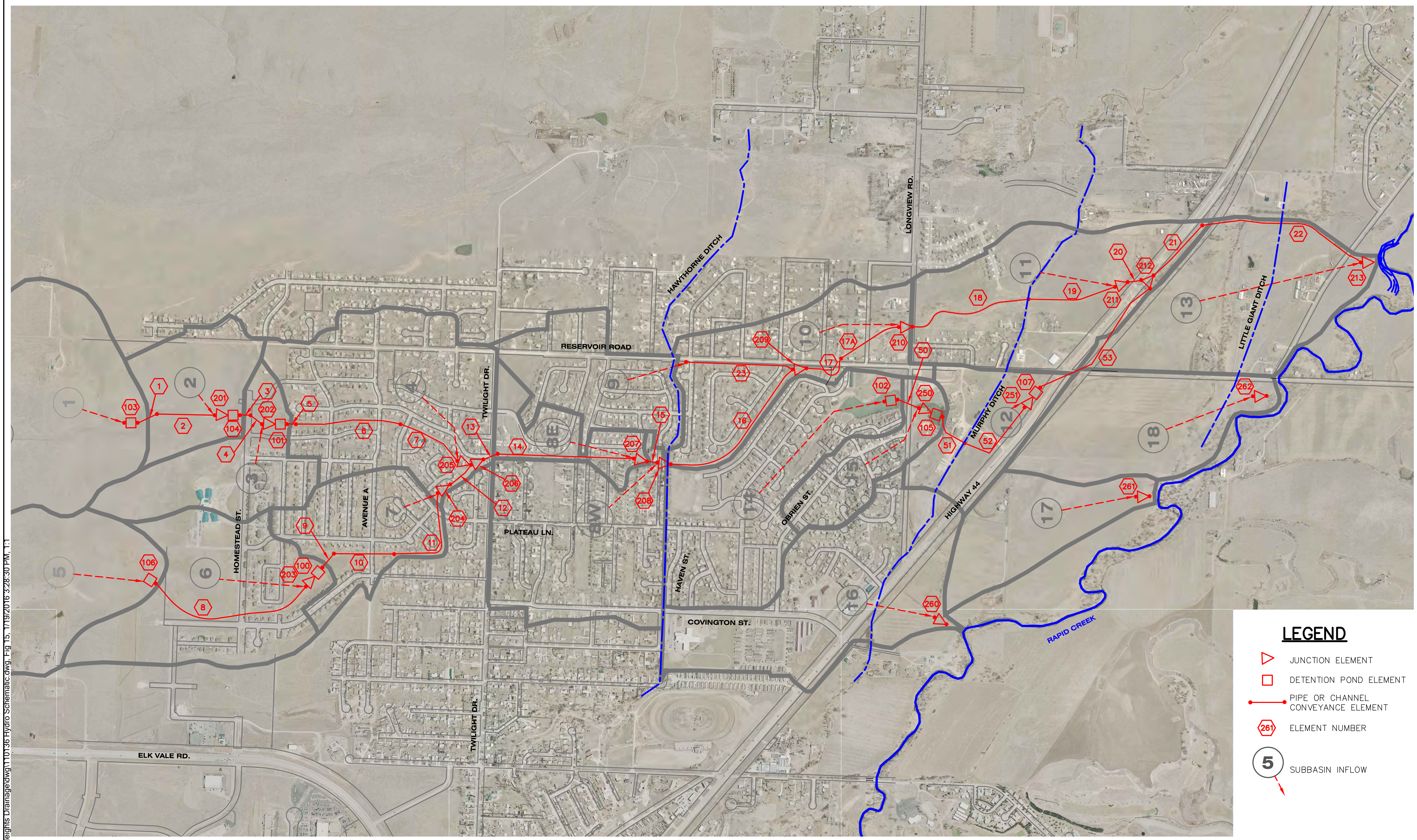
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
DBDPA
HYDROLOGIC
ROUTING
NETWORK

Sheet:
**FIGURE 14**




## LEGEND

- JUNCTION ELEMENT
- DETENTION POND ELEMENT
- PIPE OR CHANNEL CONVEYANCE ELEMENT
- 261 ELEMENT NUMBER
- 5 SUBBASIN INFLOW

0 300' 600' 1200'

SCALE: 1"=1200'

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|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

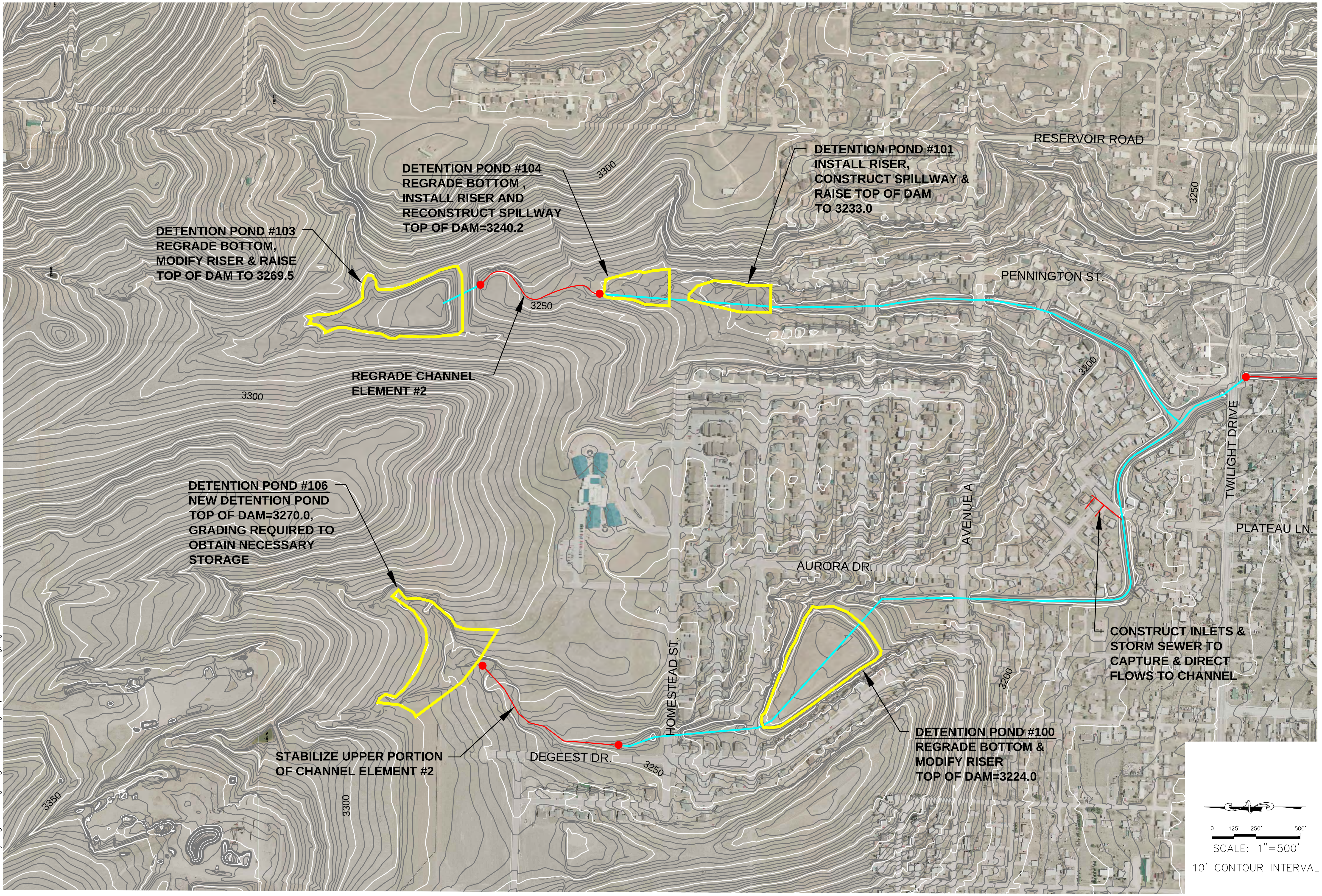
COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: DBDPA HYDROLOGIC ROUTING NETWORK WITH AERIAL

Sheet: FIGURE 15

H:\110136 County Heights Drainage\dwg\110136 Hydro Schematic.dwg, Fig 15, 1/19/2016 3:28:30 PM, 1:1




H:\110136 County Heights Drainage\dwg\110136 Drainage Improvements.dwg, Fig 16, 1/19/2016 3:31:27 PM, 1:1

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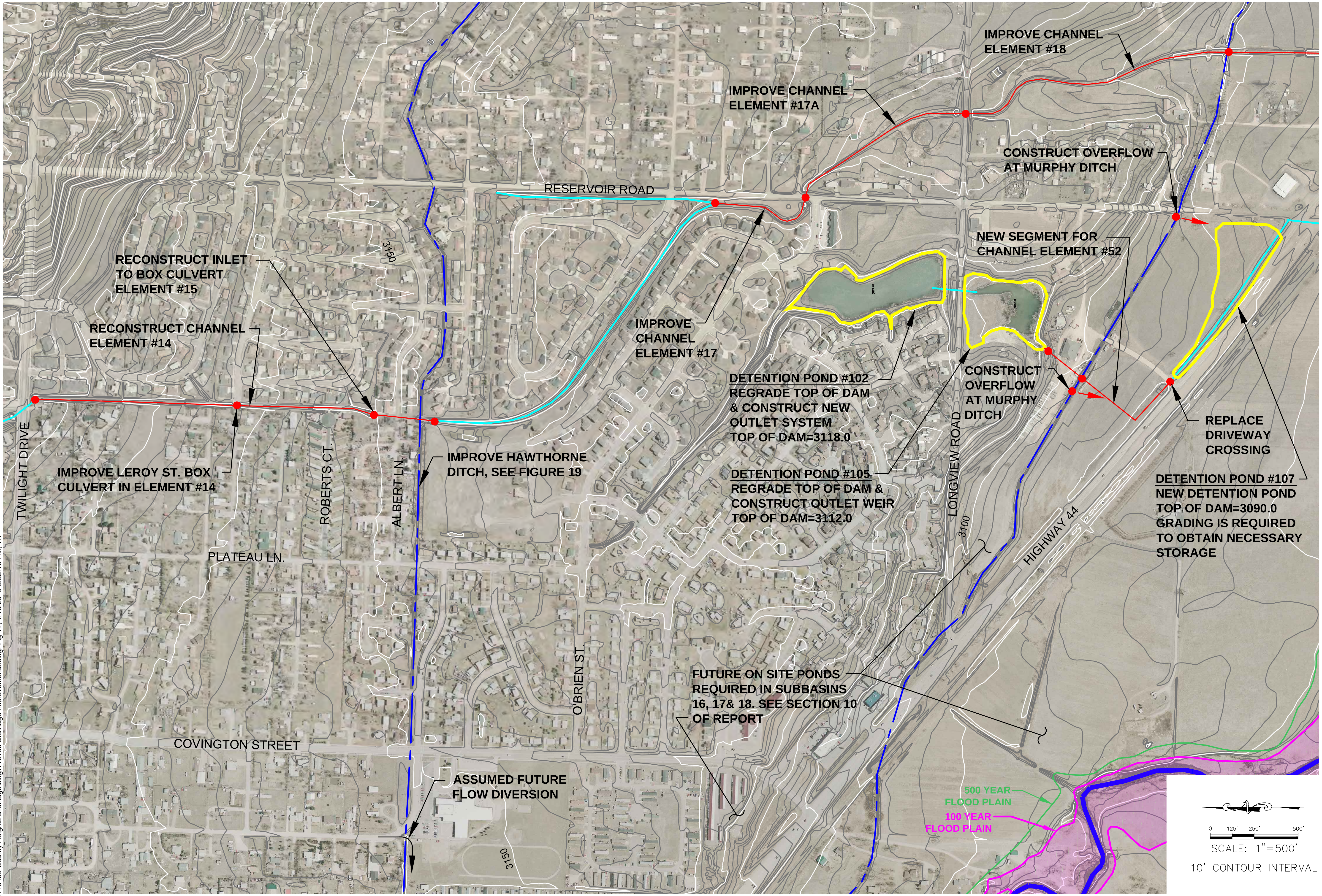
| | |
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
ENLARGED SITE MAP (NORTH)

Sheet:
FIGURE 16




H:\110136 County Heights Drainage\dwg\110136 Drainage Improvements.dwg, Fig 17, 1/19/2016 3:32:16 PM, 1:1

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Engineering Division

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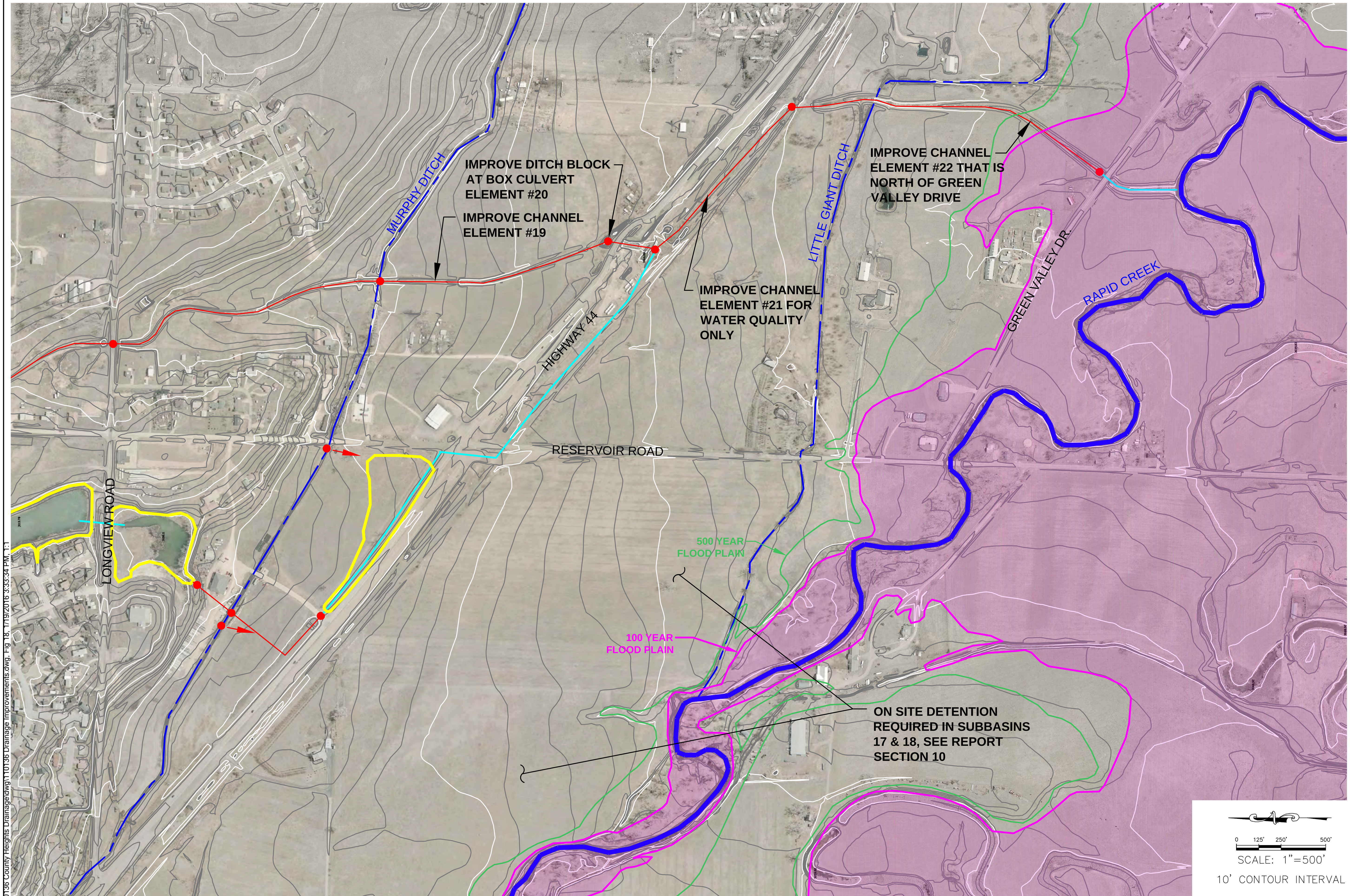
| | |
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
ENLARGED SITE MAP (MIDDLE)

Sheet:
FIGURE 17




H:\110136 County Heights Drainage\dwg\110136 Drainage Improvements.dwg, Fig 18, 1/19/2016 3:33:34 PM, 1:1

Prepared For:
Public Works Department
Engineering Division
RAPID CITY SOUTH DAKOTA INCORPORATED 1882

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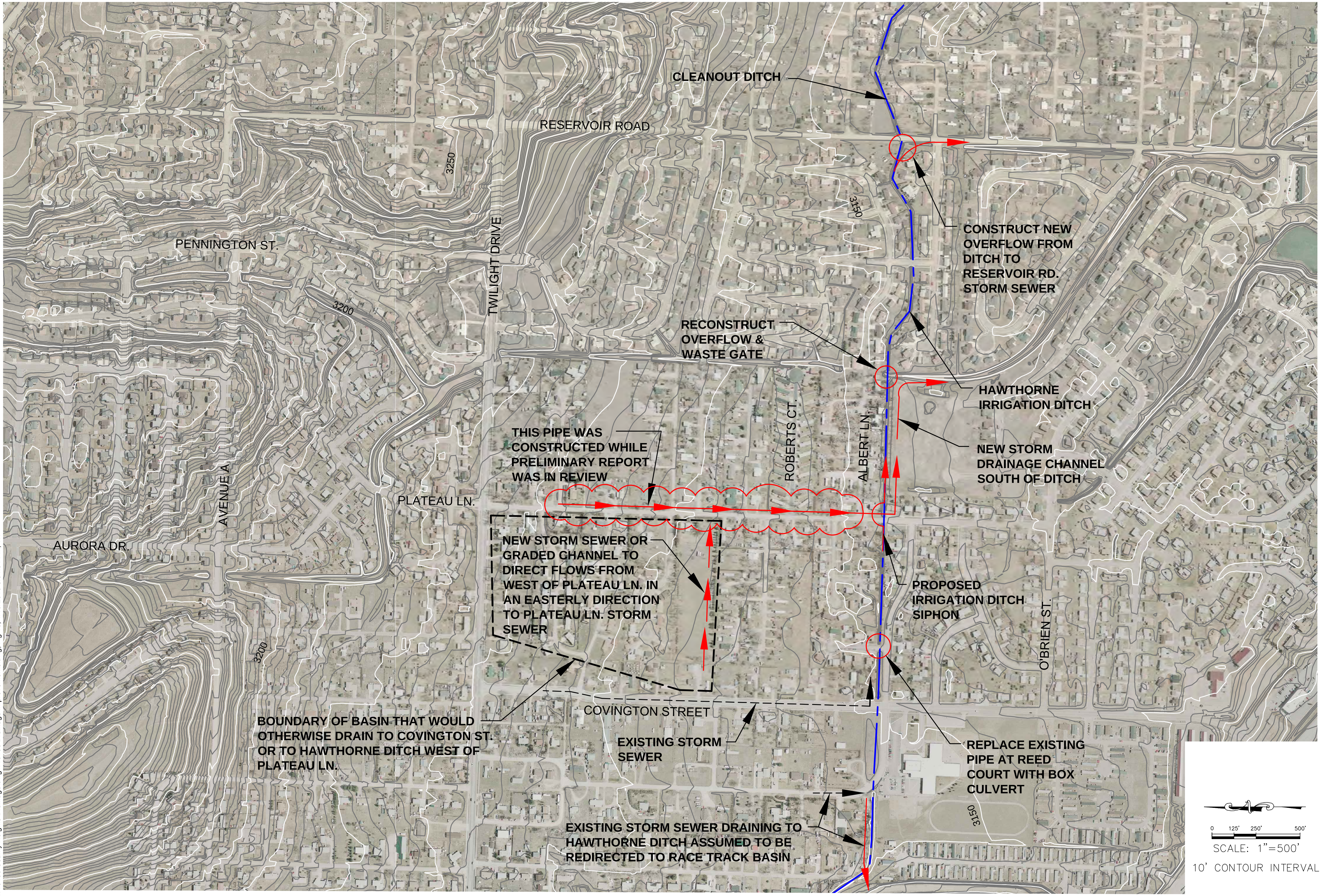
| | |
|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title:
ENLARGED SITE MAP (SOUTH)

Sheet:
**FIGURE 18**




F:\110136 County Heights Drainage\dwg\110136 Drainage Improvements.dwg, Fig 19, 1/19/2016 3:34:54 PM, 1:1

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|---|---|
| Scale: 1"=1500' | |
| Designed By: JDF | Drawn By: JRK |
| Design Date: SEPT. 2012 | Print Date: JAN. 2016 |
| Internal Job No: 110136 | |
| Surveyed By: | Survey Date: |
| Revisions: | |

COUNTY HEIGHTS DRAINAGE BASIN
DESIGN PLAN AMENDMENT

PROJECT # DR10-1870, CIP No. 50757

Sheet Title: HAWTHORNE DITCH RECOMMENDATIONS

Sheet: FIGURE 19

# APPENDIX A

DATA AND PRINTOUTS

FOR

## EXISTING LAND USE
## AND
## EXISTING HYDRAULIC CONDITIONS

The following input data tables are included in Appendix A:

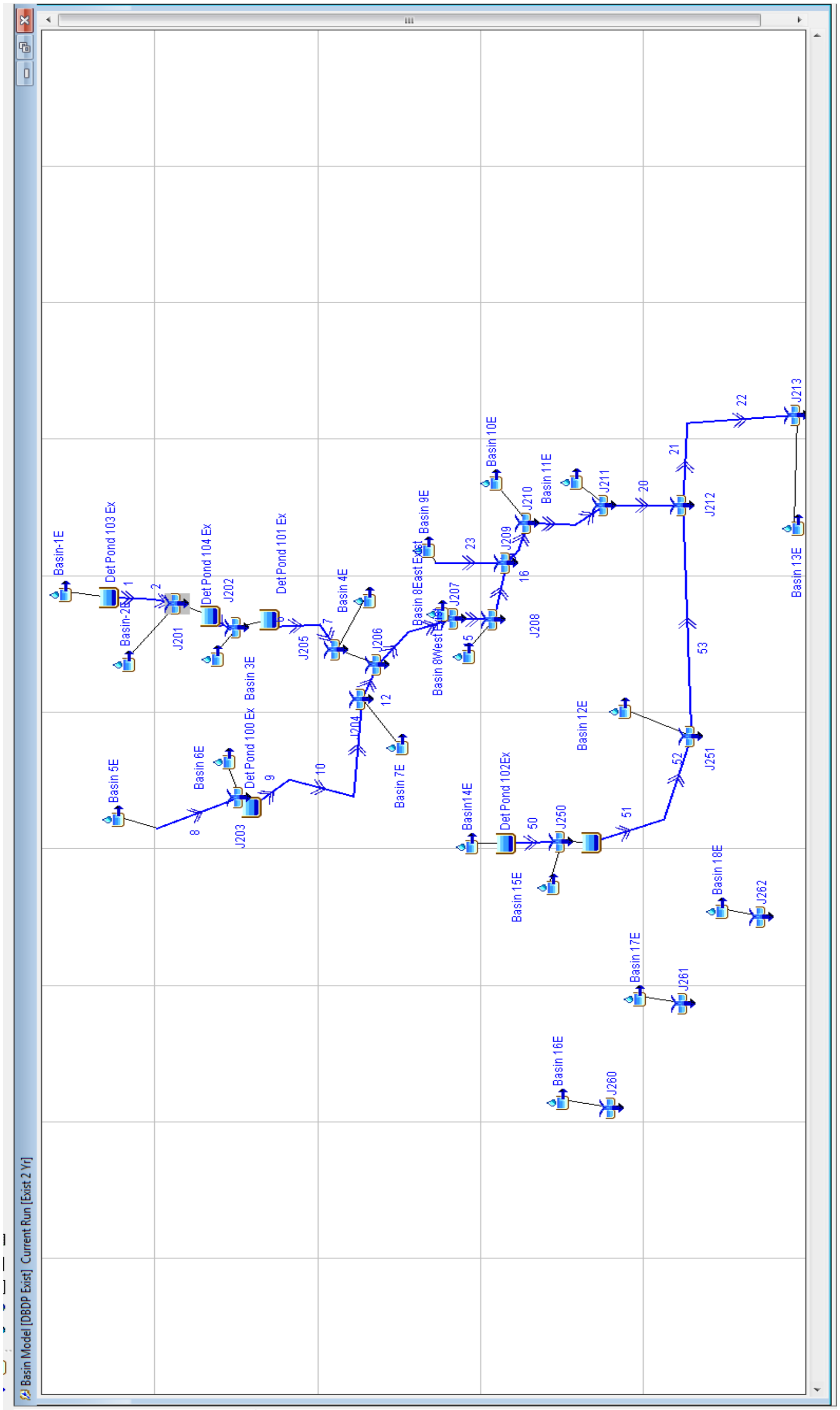
- A-1 SUMMARY OF BASIN INPUT DATA EXISTING LAND USE CONDITIONS
- A-2 GREEN AND AMPT LOSS DATA EXISTING AND FUTURE LAND USE CONDITIONS
- A-3 EXISTING LAND USE IMPERVIOUSNESS
- A-4 LAG TIME FOR EXISTING CONDITIONS
- DETENTION POND STAGE STORAGE DISCHARGE CURVES

The following direct printouts from HMS are included in Appendix A:

- 2 Year HMS Summary Output Table
- 10 Year HMS Summary Output Table
- 100 Year HMS Summary Output Table

***Users of this report need to be aware that the HMS program routes only flows entering the upstream end of the element and ignores the possibility that any adjacent sub-basin flow may be entering the element. Due to this limitation the user must exercise caution when using Model calculated peak channel and pipe flows. Flows for design purposes must be increased appropriately using engineering judgement or other suitable method to account for incoming sub-basin flows.***

*(Note: HMS Printout date of September 2012 is correct as this was that date the original review submittal for the report was made to the City. Multiyear review period ensured and HMS model was not rerun between that date and the time of this Final Report preparation)*



Basin Model [DBDP Exist] Current Run [Exist 2 Yr]
Basin-1E
Det Pond 103 Ex
J201
Det Pond 104 Ex
J202
Basin 3E
Det Pond 101 Ex
J205
Basin 4E
J206
J204
Basin 7E
Basin 5E
Basin 6E
Det Pond 100 Ex
J203
Basin 8West
J207
J208
Basin 9E
J209
Basin 10E
J210
Basin 11E
J211
J212
J213
Basin 13E
Basin 12E
J251
Basin14E
Det Pond 102Ex
J250
Basin 15E
Basin 18E
J262
Basin 17E
J261
Basin 16E
J260

TABLE A-1

COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

SUMMARY OF BASIN INPUT DATA

EXISTING LAND USE CONDITIONS

| SUBBASIN NUMBER | AREA (SQ MI) | AREA (ACRES) | INITIAL CONTENT | SATURATED CONTENT | SUCTION (IN) | CONDUCTIVITY (IN/HR) | IMPERVIOUS (EIA) % | INITIAL STORAGE % | MAX STORAGE (IN) | LAG TIME (HR) | PEAKING COEF |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Basin-1E | 0.31 | 198 | 0.116 | 0.434 | 3.500 | 0.512 | 0.9 | 0 | 0.40 | 0.47 | 0.7 |
| Basin-2E | 0.08 | 51 | 0.116 | 0.434 | 3.500 | 0.512 | 0.9 | 0 | 0.40 | 0.27 | 0.7 |
| Basin 3E | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 19.1 | 0 | 0.37 | 0.34 | 0.6 |
| Basin 4E | 0.19 | 122 | 0.116 | 0.434 | 3.500 | 0.512 | 25.6 | 0 | 0.35 | 0.38 | 0.6 |
| Basin 5E | 0.25 | 160 | 0.116 | 0.434 | 3.539 | 0.510 | 0.9 | 0 | 0.40 | 0.56 | 0.7 |
| Basin 6E | 0.15 | 96 | 0.116 | 0.434 | 3.500 | 0.512 | 16.3 | 0 | 0.37 | 0.42 | 0.6 |
| Basin 7E | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 27.8 | 0 | 0.35 | 0.29 | 0.6 |
| Basin 8Ex EAST | 0.04 | 26 | 0.116 | 0.434 | 3.500 | 0.512 | 26.3 | 0 | 0.35 | 0.36 | 0.6 |
| Basin 8Ex WEST | 0.16 | 102 | 0.116 | 0.434 | 3.500 | 0.512 | 22.9 | 0 | 0.35 | 0.94 | 0.5 |
| Basin 9E | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 27.6 | 0 | 0.35 | 0.30 | 0.6 |
| Basin 10E | 0.14 | 90 | 0.116 | 0.364 | 3.500 | 0.512 | 22.5 | 0 | 0.37 | 0.71 | 0.6 |
| Basin 11E | 0.18 | 115 | 0.138 | 0.346 | 6.219 | 0.264 | 5.7 | 0 | 0.40 | 0.58 | 0.6 |
| Basin 12E | 0.08 | 51 | 0.144 | 0.420 | 6.837 | 0.205 | 14.8 | 0 | 0.38 | 0.32 | 0.6 |
| Basin 13E | 0.17 | 109 | 0.169 | 0.434 | 10.459 | 0.081 | 0.9 | 0 | 0.40 | 0.73 | 0.6 |
| Basin14E | 0.17 | 109 | 0.116 | 0.434 | 3.500 | 0.512 | 24.9 | 0 | 0.35 | 0.69 | 0.6 |
| Basin 15E | 0.06 | 38 | 0.116 | 0.434 | 3.500 | 0.512 | 24.9 | 0 | 0.35 | 0.41 | 0.6 |
| Basin 16E | 0.19 | 122 | 0.133 | 0.399 | 5.610 | 0.325 | 26.5 | 0 | 0.37 | 0.50 | 0.6 |
| Basin 17E | 0.09 | 58 | 0.158 | 0.331 | 8.664 | 0.079 | 0.9 | 0 | 0.40 | 1.08 | 0.5 |
| Basin 18E | 0.13 | 83 | 0.157 | 0.350 | 8.613 | 0.105 | 0.9 | 0 | 0.40 | 0.76 | 0.5 |

TABLE A2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 1

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 15.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 80.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ZnD | Loam | 105.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 201 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

0.116

SUBBASIN 2

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 3.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 15.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 0.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 28.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 48 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 =

fc/2 = 0.116

SUBBASIN 3

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 30.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 12.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 1.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 13.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 59 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 = 0.116

SUBBASIN 4

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 2.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 40.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 34 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 7.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 36.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 121 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 = 0.116

TABLE A2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 5

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 11.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| NuC | Loam | 41.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 41.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ZnD | Loam | 64.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ScE | Clay | 0.7 | 0.012 | 0.024 | 12.453 | 0.475 | 0.385 | 0.378 | 0.196 | D |
| Weighted Average for Subbasin | | 160 | 0.255 | 0.510 | 3.539 | | 0.434 | 0.233 | 0.317 | |
| | | | | | | | fc/2 = | 0.116 | | |

\

SUBBASIN 6

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 38.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| NdE | Loam(1) | 30.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 3.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 27.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 99 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 7

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 7.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 14.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 21.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 10.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 55 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | | 0.116 | | |

SUBBASIN 8E

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Nde | Loam(1) | 1.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 18.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 6.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 26 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

TABLE A2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 8W

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SkA | Loam(3) | 102 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 0.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 102 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 9

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Nde | Loam(1) | 5.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 0.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| SkA | Loam(3) | 29.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 21.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 58 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 10

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 8.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 55.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 18.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 6.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 89 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 11

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Eg | Silty Clay Loam | 1 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| SeA | Loam | 11.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 5.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay Loam | 63.1 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 31.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 112 | 0.132 | 0.264 | 6.219 | | 0.364 | 0.277 | 0.225 | |
| | | | | | | | fc/2 = | 0.138 | | |

TABLE A2
COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 12

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SkB | Loam(2) | 3.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay Loam | 34.5 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 11.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 49 | 0.103 | 0.205 | 6.837 | | 0.346 | 0.287 | 0.202 | |
| | | | | | | | fc/2 = | 0.144 | | |

SUBBASIN 13

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Bp | Silt Loam | 0.5 | 0.134 | 0.268 | 6.567 | 0.501 | 0.486 | 0.284 | 0.344 | B |
| Co | Loam | 0.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 96 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 10.3 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 107 | 0.040 | 0.081 | 10.459 | | 0.420 | 0.338 | 0.251 | |
| | | | | | | | fc/2 = | 0.169 | | |

SUBBASIN 14

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 13.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 49.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 43.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 106 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 15

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 6.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 23.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 6.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 36 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

TABLE A2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 16

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Eg | Silty Clay Loam | 0.9 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| EnD | Silt Loam | 18.1 | 0.134 | 0.268 | 6.567 | 0.501 | 0.486 | 0.284 | 0.344 | D |
| SeA | Loam | 45.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 4.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 0.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay loam | 41.3 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 10.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 122 | 0.163 | 0.325 | 5.610 | | 0.399 | 0.267 | 0.266 | |
| | | | | | | | fc/2 = | 0.133 | | |

SUBBASIN 17

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Co | Loam | 0.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 9.8 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 44.8 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 55 | 0.039 | 0.079 | 8.664 | | 0.331 | 0.316 | 0.173 | |
| | | | | | | | fc/2 = | 0.158 | | |

SUBBASIN 18

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Co | Loam | 5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 21.7 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 52.8 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 80 | 0.053 | 0.105 | 8.613 | | 0.350 | 0.314 | 0.194 | |
| | | | | | | | fc/2 = | 0.157 | | |

(1) NdE is Gravelly Loam - Assume as Loam

(2) SkB is Santanta Urban Land Comples - Assumed as Loam and as Type B

(3) SkA is Santanta Urban Land Comples - Assumed as Loam and as Type B

* Bare soil hydraulic conductivity values from City of Rapid City Drainage Manual

** Vegetated soil hydraulic conductivity values assumed at 2 times bare soil.

TABLE A-3 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT
EXISTING CONDITION BASIN IMPERVIOUSNESS

| BASIN #1E | XX | XX | XX | XX | Open | XX | XX | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 0 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 201 | 0 | 0 | 201 | 2.0 | 0.3 | 0.9 |

| BASIN #2E | | | | | Open | | | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | Space | XX | XX | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 0 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 48 | 0 | 0 | 48 | 2.0 | 0.3 | 0.9 |

| BASIN #3E | | | | | Open | | | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | MDR | LDR | Space | School | XX | AREA | % | % | % |
| % Impervious | 0 | 0 | 60 | 34 | 2 | 50 | 0 | | | | |
| Acres | 0 | 0 | 6.5 | 18.7 | 25.6 | 8 | 0 | 59 | 25.1 | 12.6 | 19.1 |

| BASIN #4E | XX | XX | XX | XX | XX | LDR | XX | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 32 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 120.5 | 0 | 121 | 32.0 | 18.1 | 25.6 |

| BASIN #5E | XX | XX | XX | XX | XX | XX | Open | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Space | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 0 | 2 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 0 | 160 | 160 | 2.0 | 0.3 | 0.9 |

| BASIN #6E | XX | XX | MDR | LDR | Open | XX | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Imp | 0 | 0 | 60 | 36 | 2 | 0 | 50 | | | | |
| Acres | 0 | 0 | 20 | 12.7 | 58.6 | 0 | 8.1 | 99 | 21.9 | 10.3 | 16.3 |

TABLE A-3 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT
EXISTING CONDITION BASIN IMPERVIOUSNESS

| BASIN #7E | XX | XX | XX | XX | Open | LDR | MDR | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 35 | 60 | | | | |
| Acres | 0 | 0 | 0 | 0 | 2.5 | 50.6 | 1.7 | 55 | 34.3 | 20.1 | 27.8 |

| BASIN #8Ex East | XX | XX | XX | LDR | PUD | Open | MHP | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Space | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 31 | 48 | 2 | 35 | | | | |
| Acres | 0 | 0 | 0 | 6.4 | 12.6 | 6 | 1 | 26 | 32.7 | 18.7 | 26.3 |

| BASIN #8Ex West | XX | XX | XX | XX | Open | LDR | MHP | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 31 | 35 | | | | |
| Acres | 0 | 0 | 0 | 0 | 11 | 57 | 34 | 102 | 29.2 | 15.8 | 22.9 |

| BASIN #9E | XX | XX | XX | XX | Open | LDR | PUD | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 32 | 48 | | | | |
| Acres | 0 | 0 | 0 | 0 | 3.5 | 40.2 | 13.9 | 58 | 34.0 | 19.9 | 27.6 |

| BASIN #10E | XX | XX | XX | Open | LI | LDR | PUD | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Space | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 2 | 80 | 30 | 35 | | | | |
| Acres | 0 | 0 | 0 | 11 | 0.8 | 47.3 | 30.3 | 89 | 28.7 | 15.4 | 22.5 |

| BASIN #11E | XX | XX | XX | HWY 44 | Open | LDR | PUBLIC | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | ROW | Space | | FIRE STA | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 32 | 2 | 32 | 80 | | | | |
| Acres | 0 | 0 | 0 | 14.5 | 91.2 | 3 | 3.5 | 112 | 9.1 | 2.8 | 5.7 |

TABLE A-3 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT
EXISTING CONDITION BASIN IMPERVIOUSNESS

| BASIN #12E | XX | XX | HWY 44 | Open | MDR | XX | LI | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | ROW | Space | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 32 | 2 | 60 | 0 | 80 | | | | |
| Acres | 0 | 0 | 11 | 28.4 | 8 | 0 | 1.2 | 49 | 20.3 | 9.1 | 14.8 |

| BASIN #13E | | | | | | | Open | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | XX | HWY 44 | Space | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | ROW | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 2 | 2 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 5.8 | 101.2 | 107 | 2.0 | 0.3 | 0.9 |

| BASIN #14E | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | PUD | LDR | Open | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | | Space | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 36 | 34 | 2 | | | | |
| Acres | 0 | 0 | 0 | 0 | 34 | 60.4 | 11.1 | 106 | 31.3 | 17.5 | 24.9 |

| BASIN #15E | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | Open | PUD | LI | MDR | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | Space | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 2 | 34 | 80 | 60 | | | | |
| Acres | 0 | 0 | 0 | 6 | 27.5 | 0.9 | 2 | 36 | 31.3 | 17.5 | 24.9 |

| BASIN #16E | GC | Hwy 44 | MHP | LDR | PUD | Open | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ROW | | | | Space | | AREA | % | % | % |
| % Impervious | 95 | 33 | 40 | 35 | 51 | 2 | 50 | | | | |
| Acres | 1.5 | 16.8 | 0.5 | 8.2 | 48.3 | 37.2 | 9.3 | 122 | 32.9 | 18.9 | 26.5 |

| BASIN #17 | | | | | | Open | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | XX | Space | XX | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 2 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 55 | 0 | 55 | 2.0 | 0.3 | 0.9 |

TABLE A-3 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT
EXISTING CONDITION BASIN IMPERVIOUSNESS

| BASIN #18 | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | Hwy 44 | Open | XX | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | ROW | Space | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 2 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 3.1 | 76.9 | 0 | 80 | 2.0 | 0.3 | 0.9 |

XX = This column not used

LI = Light Industrial

GC = General Commercial

NC = Neighborhood Commerical

MDR = Multi Unit (Impervious = Average of detached and attached)

LDR/PRD 1.5 U/Ac = Low Density Residential with Planned Development, Density of 1.5 unit per acre

LDR = Low Density Residential, Imperviouse based Curve Developed within study area by FMG
Existing Units/Acre Measured From Aerials, Future Developed Assumes 3.4 units/acre

OFFICE WITH PCD = Office Commerial with Planned Development

SCHOOL = Public School

MHP = Mobile Home Park, Impervious assumed at 40% based on estimates of existing parks

PUD = Planned Unit Development, Imperviouse based on combination existing and proposed uses in PUD

HWY 44 ROW = Highway 44 right of way, including railroad right of way, imperviouse based on existing pavements

PUBLIC FIRE STA = Public Land used for fire station, impervious assumed same as light industrial

PUBLIC OR VERY LOW LDR = Impervious based on parks/cemeteries or 0.5 housing units/acre

OPEN SPACE = Open space/undeveloped at Existing Ponds 100, 101, 103, and 104

TOTAL AREA = Total size of drainage basin

MIA = Mapped Imperviouse Area in Percent

EIA AVE = Effective Imperviouse Area in percent based on Average Sutherland Equation (Included in Table for comparision only

**EIA HD = Effective Imperviouse Area in percent based on Highly Connected Sutherland Equation (This equation used in study for EIA)**

TABLE A-4 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT
TABLE OF INFORMATION FOR DETERMINING LAG TIME - EXISTING LAND USE DBDP CONDITIONS

| Basin # | Area (Sq Mi) | Area (Ac) | Exist Mapped Imperv (%) | I (ft) | L (Mi) | lc (ft) | Lc (Mi) | A Soils CN 39 (Ac) | B Soils CN 61 (Ac) | C Soils CN 74 (Ac) | D Soils CN 80 (Ac) | Total Area (Ac) | Ave CN For Grass | Overall Ave CN Including MIA | S Max Pot Retention (in) | 10' Contour Lengths (ft) | Y Average and Slop (%) | **Calculated NRCS Lag Time Used In Report (Hours)** | Comparison Lag Time From Snyder Equation Using Ct = .6 (Hours) | Comparison Snyder Ct Coef. Back Calculated From NRCS Lag Time* | Comparison Lag Time Calculated With Snyder Eq. and Ct from USACE Method (Hours) | Comparison Snyder Ct Coef Calculated From USACE Equation** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.31 | 201 | 2.0 | 3855 | 0.73 | 1907 | 0.36 | 0 | 185 | 16 | 0 | 201 | 62.0 | 62.7 | 5.941 | 91178 | 10.41 | **0.47** | 0.40 | 0.70 | 3.05 | 4.55 |
| 2 | 0.08 | 48 | 2.0 | 1849 | 0.35 | 637 | 0.12 | 0 | 44 | 4 | 0 | 48 | 62.1 | 62.9 | 5.910 | 20771 | 9.93 | **0.27** | 0.23 | 0.69 | 1.76 | 4.55 |
| 3 | 0.09 | 59 | 25.1 | 2973 | 0.56 | 1057 | 0.20 | 0 | 27 | 32 | 0 | 59 | 68.1 | 75.6 | 3.227 | 16643 | 6.48 | **0.34** | 0.31 | 0.66 | 0.33 | 0.63 |
| 4 | 0.19 | 121 | 32.0 | 3798 | 0.72 | 1537 | 0.29 | 0 | 85 | 36 | 0 | 121 | 64.9 | 75.5 | 3.250 | 39944 | 7.58 | **0.38** | 0.38 | 0.61 | 0.33 | 0.52 |
| 5 | 0.25 | 160 | 2.0 | 4662 | 0.88 | 2191 | 0.41 | 0 | 106 | 53 | 1 | 160 | 65.4 | 66.1 | 5.135 | 56975 | 8.17 | **0.56** | 0.44 | 0.76 | 3.37 | 4.55 |
| 6 | 0.15 | 99 | 21.9 | 3664 | 0.69 | 1629 | 0.31 | 0 | 60 | 39 | 0 | 99 | 66.1 | 73.1 | 3.685 | 29887 | 6.93 | **0.42** | 0.38 | 0.66 | 0.44 | 0.70 |
| 7 | 0.09 | 55 | 34.3 | 2669 | 0.51 | 1268 | 0.24 | 0 | 47 | 8 | 0 | 55 | 62.8 | 74.9 | 3.356 | 18501 | 7.72 | **0.29** | 0.32 | 0.55 | 0.26 | 0.50 |
| 8E | 0.04 | 26 | 32.7 | 3100 | 0.59 | 1139 | 0.22 | 0 | 26 | 0 | 0 | 26 | 61.0 | 73.1 | 3.680 | 8183 | 7.23 | **0.36** | 0.32 | 0.67 | 0.26 | 0.49 |
| 8W | 0.16 | 102 | 29.2 | 4770 | 0.90 | 1982 | 0.38 | 0 | 102 | 0 | 0 | 102 | 61.0 | 71.8 | 3.927 | 9984 | 2.25 | **0.94** | 0.43 | 1.30 | 0.41 | 0.56 |
| 9 | 0.09 | 58 | 34.0 | 2284 | 0.43 | 978 | 0.19 | 0 | 57 | 1 | 0 | 58 | 61.2 | 73.7 | 3.569 | 15265 | 6.04 | **0.30** | 0.28 | 0.64 | 0.23 | 0.50 |
| 10 | 0.14 | 89 | 28.7 | 4285 | 0.81 | 1769 | 0.34 | 0 | 89 | 0 | 0 | 89 | 61.0 | 71.6 | 3.963 | 12898 | 3.33 | **0.71** | 0.41 | 1.05 | 0.39 | 0.57 |
| 11 | 0.18 | 112 | 9.1 | 4343 | 0.82 | 2500 | 0.47 | 0 | 48 | 63 | 1 | 112 | 68.5 | 71.2 | 4.049 | 25247 | 5.17 | **0.58** | 0.45 | 0.78 | 1.05 | 1.40 |
| 12 | 0.08 | 49 | 20.3 | 2760 | 0.52 | 1402 | 0.27 | 0 | 14 | 35 | 0 | 49 | 70.3 | 75.9 | 3.173 | 13386 | 6.27 | **0.32** | 0.33 | 0.58 | 0.41 | 0.75 |
| 13 | 0.17 | 107 | 2.0 | 5047 | 0.96 | 2852 | 0.54 | 0 | 1 | 10 | 96 | 107 | 79.3 | 79.6 | 2.557 | 12166 | 2.61 | **0.73** | 0.49 | 0.89 | 3.73 | 4.55 |
| 14 | 0.17 | 106 | 31.3 | 4501 | 0.85 | 2215 | 0.42 | 0 | 106 | 0 | 0 | 106 | 61.0 | 72.6 | 3.778 | 16883 | 3.66 | **0.69** | 0.44 | 0.94 | 0.39 | 0.53 |
| 15 | 0.06 | 36 | 31.3 | 2755 | 0.52 | 1447 | 0.27 | 0 | 36 | 0 | 0 | 36 | 61.0 | 72.6 | 3.778 | 7464 | 4.76 | **0.41** | 0.33 | 0.73 | 0.30 | 0.53 |
| 16 | 0.19 | 122 | 32.9 | 4512 | 0.85 | 1648 | 0.31 | 0 | 62 | 41 | 19 | 122 | 68.4 | 78.1 | 2.802 | 26631 | 5.01 | **0.50** | 0.40 | 0.75 | 0.34 | 0.51 |
| 17 | 0.09 | 55 | 2.0 | 3415 | 0.65 | 1711 | 0.32 | 0 | 0 | 45 | 10 | 55 | 75.1 | 75.5 | 3.243 | 1928 | 0.80 | **1.08** | 0.38 | 1.73 | 2.85 | 4.55 |
| 18 | 0.13 | 80 | 2.0 | 3820 | 0.72 | 2058 | 0.39 | 0 | 5 | 53 | 22 | 80 | 74.8 | 75.3 | 3.283 | 6958 | 2.00 | **0.76** | 0.41 | 1.11 | 3.11 | 4.55 |

*Note: Ct back calculation is described as dividing the calculated NRCS Lag Time by the Synder Equation to find Ct
**USACE Equation is Ct = 7.81/(i^0.78) where I is imperviousness
Synder Equation is per RCIDCM: tp = Lag Time = Ct(LLc)^0.3

**DETENTION POND 100**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 18" RCP FL 3210 (CFS) | 48" Dia Riser Weir Rim 3220 (CFS) | 48" Dia Riser Orf Rim 3220 (CFS) | Total Structure Flow (CFS) | 60' Spillway 3222.5 (CFS) | 36"" Dia RCP Outlet Cap. (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|---|---|---|---|
| 3210 | 0 | 0 | 0 | 0 | | | 0 | | 32 | 0 |
| 3211 | 10713 | 0.12 | 0.12 | 3 | | | 3 | | 47 | 3 |
| 3212 | 39987 | 0.58 | 0.70 | 9 | | | 9 | | 55 | 9 |
| 3213 | 81428 | 1.39 | 2.10 | 13 | | | 13 | | 62 | 13 |
| 3214 | 114174 | 2.25 | 4.34 | 16 | | | 16 | | 68 | 16 |
| 3215 | 140664 | 2.93 | 7.27 | 18 | | | 18 | | 74 | 18 |
| 3216 | 163666 | 3.49 | 10.76 | 21 | | | 21 | | 80 | 21 |
| 3217 | 180665 | 3.95 | 14.71 | 22 | | | 22 | | 85 | 22 |
| 3218 | 191097 | 4.27 | 18.98 | 24 | | | 24 | | 90 | 24 |
| 3219 | 199498 | 4.48 | 23.47 | 25 | | | 25 | | 95 | 25 |
| 3220 | 207842 | 4.68 | 28.14 | 27 | 0 | 0 | 27 | | 99 | 27 |
| 3221 | 215938 | 4.86 | 33.01 | 29 | 41 | ~~61~~ | 70 | | 104 | 70 |
| 3222 | 226740 | 5.08 | 38.09 | 30 | ~~117~~ | 85 | 115 | | 108 | 108 |
| 3222.5 | 230000 | 2.62 | 40.71 | 30 | ~~163~~ | 95 | 125 | 0 | 110 | 110 |
| 3223 | 234906 | 2.67 | 43.38 | 31 | ~~215~~ | 104 | 135 | 55 | 112 | 167 |
| 3224 | 243972 | 5.50 | 48.87 | 32 | ~~331~~ | 121 | 153 | 335 | 116 | 451 |

Contour Area digitized from original design plans for Pond 100

**DETENTION POND 101**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 36" RCP 3221.2 (CFS) | 20' Spillway 3230 (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|
| 3221.2 | 0 | 0 | 0 | 0 | | 0 |
| 3222 | 11042 | 0.10 | 0.10 | 3 | | 3 |
| 3223 | 21834 | 0.38 | 0.48 | 13 | | 13 |
| 3224 | 29895 | 0.59 | 1.07 | 28 | | 28 |
| 3225 | 32960 | 0.72 | 1.79 | 46 | | 46 |
| 3226 | 36037 | 0.79 | 2.59 | 60 | | 60 |
| 3227 | 39242 | 0.86 | 3.45 | 70 | | 70 |
| 3228 | 42494 | 0.94 | 4.39 | 78 | | 78 |
| 3229 | 45968 | 1.02 | 5.40 | 86 | | 86 |
| 3230 | 49421 | 1.09 | 6.50 | 92 | 0 | 92 |
| 3231 | 53015 | 1.18 | 7.67 | 99 | 65 | 164 |
| 3232 | 56636 | 1.26 | 8.93 | 105 | 195 | 300 |
| 3232.5 | 59346 | 0.67 | 9.60 | 108 | 280 | 388 |

Contour Area digitized from original design plans for Pond 101

**DETENTION POND 102**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 6" Orifice 3113.8 (CFS) | Elev 3118 Estimate 240' Top Dam (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|
| 3113.8 | 148100 | 0 | 0 | 0.0 | | 0 |
| 3114 | 156820 | 0.70 | 0.70 | 0.1 | | 0.1 |
| 3116 | 182950 | 7.80 | 8.50 | 1.3 | | 1.3 |
| 3118 | 209080 | 9.00 | 17.50 | 1.9 | | 1.9 |
| 3119 | 222000 | 4.95 | 22.45 | 2.1 | 720 | 722 |

Contour Area digitized from gis, top dam and water verified+-  by field survey

## DETENTION POND 103
## ELEVATION - STORAGE - DISCHARGE DATA
### COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 18" RCP FL 3252 (CFS) | 30" RCP FL 3258 (CFS) | 60" Dia Riser Weir Rim 3265 (CFS) | 60" Dia Riser Orf Rim 3265 (CFS) | Total Structure Flow (CFS) | 10' Spillway 3266.5 (CFS) | 42"Dia RCP Outlet Cap. (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3252 | 0 | 0 | 0 | 0 | | | | 0 | | 0 | 0 |
| 3253 | 11979 | 0.14 | 0.14 | 3 | | | | 3 | | 9 | 3 |
| 3254 | 47050 | 0.68 | 0.82 | 9 | | | | 9 | | 26 | 9 |
| 3256 | 70251 | 2.69 | 3.51 | 15 | | | | 15 | | 74 | 15 |
| 3258 | 87679 | 3.63 | 7.13 | 20 | 0 | | | 20 | | 108 | 20 |
| 3260 | 102877 | 4.37 | 11.51 | 23 | 14 | | | 37 | | 132 | 37 |
| 3262 | 118650 | 5.09 | 16.59 | 27 | 38 | | | 65 | | 150 | 65 |
| 3264 | 138725 | 5.91 | 22.50 | 29 | 51 | | | 80 | | 167 | 80 |
| 3265 | 147500 | 3.29 | 25.79 | 31 | 57 | 0 | 0 | 88 | | 175 | 88 |
| 3266 | 156618 | 3.49 | 29.28 | 32 | 62 | 52 | ~~94~~ | 146 | | 182 | 146 |
| 3266.5 | 161500 | 1.83 | 31.10 | 32 | 64 | 95 | ~~115~~ | 191 | 0 | 186 | 186 |
| 3267 | 167000 | 1.89 | 32.99 | 33 | 67 | ~~146~~ | 133 | 233 | 10 | 189 | 199 |
| 3268 | 177705 | 3.96 | 36.95 | 34 | 71 | ~~269~~ | 163 | 268 | 75 | 197 | 272 |

Contour Area digitized from original design plans for Pond 103

**DETENTION POND 104**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 30" RCP 3226 (CFS) | 20' Spillway 3338 (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|
| 3226 | 0 | 0 | 0 | 0 | | 0 |
| 3227 | 3817 | 0.04 | 0.04 | 4 | | 4 |
| 3228 | 10491 | 0.16 | 0.21 | 14 | | 14 |
| 3229 | 15846 | 0.30 | 0.51 | 28 | | 28 |
| 3230 | 23203 | 0.45 | 0.96 | 38 | | 38 |
| 3231 | 26326 | 0.57 | 1.53 | 44 | | 44 |
| 3232 | 29761 | 0.64 | 2.17 | 49 | | 49 |
| 3233 | 33300 | 0.72 | 2.89 | 53 | | 53 |
| 3234 | 36943 | 0.81 | 3.70 | 58 | | 58 |
| 3235 | 40611 | 0.89 | 4.59 | 62 | | 62 |
| 3236 | 44989 | 0.98 | 5.57 | 65 | | 65 |
| 3237 | 49338 | 1.08 | 6.66 | 69 | | 69 |
| 3238 | 53895 | 1.18 | 7.84 | 72 | 0 | 72 |
| 3239 | 58653 | 1.29 | 9.13 | 75 | 60 | 135 |
| 3240 | 63185 | 1.40 | 10.53 | 78 | 170 | 248 |
| 3240.2 | 64000 | 0.29 | 10.82 | 79 | 196 | 275 |

Contour Area digitized from original design plans for Pond 104

**DETENTION POND 105**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

## EXISTING POND

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | ACCUM VOL (AC-FT) | 5' Weir 3108.1 (CFS) | Approximate Ovetop 3111 (CFS) | FINAL DISCHARGE CURVE (CFS) |
|---|---|---|---|---|---|---|
| 3108.1 | 56600 | 0 | 0 | 0 | | 0 |
| 3109 | Interpolated | | 1.8 | 13 | | 13 |
| 3110 | 108900 | 3.61 | 3.61 | 39 | | 39 |
| 3111 | Interpolated | | 6.30 | 74 | | 74 |
| 3112 | 143700 | 5.80 | 9.41 | 115 | 300 | 415 |

Estimated top of dam at 3111, Assume 100 ft overtop length

Contour Area digitized from gis

Project: County Hts DBDP Exist Simulation Run: Exist 2 Yr

| | | | |
|---|---|---|---|
| Start of Run: | 01Jun2012, 00:00 | Basin Model: | DBDP Exist |
| End of Run: | 02Jun2012, 01:00 | Meteorologic Model: | 2 Year Storm |
| Compute Time: | 20Sep2012, 15:11:16 | Control Specifications: | Control 1 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 3.05 | 01Jun2012, 01:20 | 0.02 |
| 10 | 0.40 | 7.37 | 01Jun2012, 01:50 | 0.06 |
| 11 | 0.40 | 7.36 | 01Jun2012, 02:00 | 0.06 |
| 12 | 0.49 | 17.39 | 01Jun2012, 01:00 | 0.09 |
| 13 | 1.16 | 45.43 | 01Jun2012, 01:00 | 0.09 |
| 14 | 1.16 | 44.64 | 01Jun2012, 01:05 | 0.09 |
| 15 | 1.20 | 49.81 | 01Jun2012, 01:05 | 0.10 |
| 16 | 1.36 | 53.85 | 01Jun2012, 01:15 | 0.11 |
| 17 | 1.45 | 62.76 | 01Jun2012, 01:20 | 0.11 |
| 17A | 1.45 | 61.96 | 01Jun2012, 01:25 | 0.11 |
| 18 | 1.59 | 71.60 | 01Jun2012, 01:35 | 0.12 |
| 19 | 1.59 | 71.23 | 01Jun2012, 01:40 | 0.12 |
| 2 | 0.31 | 3.04 | 01Jun2012, 01:30 | 0.02 |
| 20 | 1.77 | 76.35 | 01Jun2012, 01:40 | 0.12 |
| 21 | 2.08 | 80.18 | 01Jun2012, 01:45 | 0.11 |
| 22 | 2.08 | 78.99 | 01Jun2012, 01:55 | 0.11 |
| 23 | 0.09 | 16.68 | 01Jun2012, 00:55 | 0.22 |
| 3 | 0.39 | 3.13 | 01Jun2012, 01:40 | 0.02 |
| 4 | 0.39 | 3.13 | 01Jun2012, 01:45 | 0.02 |
| 5 | 0.48 | 7.73 | 01Jun2012, 01:20 | 0.05 |
| 50 | 0.17 | 0.26 | 01Jun2012, 04:35 | 0.05 |
| 51 | 0.23 | 2.62 | 01Jun2012, 01:55 | 0.09 |
| 52 | 0.23 | 2.61 | 01Jun2012, 02:15 | 0.09 |
| 53 | 0.31 | 11.03 | 01Jun2012, 01:05 | 0.10 |
| 6 | 0.48 | 7.71 | 01Jun2012, 01:40 | 0.05 |
| 7 | 0.48 | 7.69 | 01Jun2012, 01:50 | 0.05 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 3.88 | 01Jun2012, 01:25 | 0.02 |
| 9 | 0.40 | 7.38 | 01Jun2012, 01:45 | 0.06 |
| Basin 10E | 0.14 | 11.85 | 01Jun2012, 01:20 | 0.19 |
| Basin 11E | 0.18 | 10.27 | 01Jun2012, 01:10 | 0.09 |
| Basin 12E | 0.08 | 11.30 | 01Jun2012, 00:55 | 0.15 |
| Basin 13E | 0.17 | 11.66 | 01Jun2012, 01:15 | 0.13 |
| Basin14E | 0.17 | 15.51 | 01Jun2012, 01:20 | 0.20 |
| Basin 15E | 0.06 | 8.43 | 01Jun2012, 01:00 | 0.20 |
| Basin 16E | 0.19 | 24.46 | 01Jun2012, 01:05 | 0.21 |
| Basin 17E | 0.09 | 5.39 | 01Jun2012, 01:35 | 0.20 |
| Basin 18E | 0.13 | 8.65 | 01Jun2012, 01:20 | 0.16 |
| Basin-1E | 0.31 | 5.81 | 01Jun2012, 01:00 | 0.02 |
| Basin-2E | 0.08 | 2.49 | 01Jun2012, 00:50 | 0.02 |
| Basin 3E | 0.09 | 10.97 | 01Jun2012, 00:55 | 0.15 |
| Basin 4E | 0.19 | 28.15 | 01Jun2012, 01:00 | 0.20 |
| Basin 5E | 0.25 | 3.93 | 01Jun2012, 01:05 | 0.02 |
| Basin 6E | 0.15 | 13.92 | 01Jun2012, 01:00 | 0.13 |
| Basin 7E | 0.09 | 17.28 | 01Jun2012, 00:55 | 0.22 |
| Basin 8East Exist | 0.04 | 6.31 | 01Jun2012, 00:55 | 0.21 |
| Basin 8West Exist | 0.16 | 8.77 | 01Jun2012, 01:35 | 0.19 |
| Basin 9E | 0.09 | 16.85 | 01Jun2012, 00:55 | 0.22 |
| Det Pond 100 Ex | 0.40 | 7.39 | 01Jun2012, 01:45 | 0.06 |
| Det Pond 101 Ex | 0.48 | 7.73 | 01Jun2012, 01:20 | 0.05 |
| Det Pond 102Ex | 0.17 | 0.26 | 01Jun2012, 04:35 | 0.05 |
| Det Pond 103 Ex | 0.31 | 3.05 | 01Jun2012, 01:20 | 0.02 |
| Det Pond 104 Ex | 0.39 | 3.13 | 01Jun2012, 01:40 | 0.02 |
| Det Pond 105 Ex | 0.23 | 2.62 | 01Jun2012, 01:50 | 0.09 |
| J201 | 0.39 | 3.22 | 01Jun2012, 01:30 | 0.02 |
| J202 | 0.48 | 12.56 | 01Jun2012, 01:00 | 0.05 |
| J203 | 0.40 | 14.11 | 01Jun2012, 01:00 | 0.06 |
| J204 | 0.49 | 17.48 | 01Jun2012, 00:55 | 0.09 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J205 | 0.67 | 28.20 | 01Jun2012, 01:00 | 0.09 |
| J206 | 1.16 | 45.59 | 01Jun2012, 01:00 | 0.09 |
| J207 | 1.20 | 50.25 | 01Jun2012, 01:05 | 0.10 |
| J208 | 1.36 | 54.88 | 01Jun2012, 01:05 | 0.11 |
| J209 | 1.45 | 63.78 | 01Jun2012, 01:15 | 0.11 |
| J210 | 1.59 | 73.36 | 01Jun2012, 01:25 | 0.12 |
| J211 | 1.77 | 76.57 | 01Jun2012, 01:40 | 0.12 |
| J212 | 2.08 | 80.93 | 01Jun2012, 01:40 | 0.11 |
| J213 | 2.25 | 84.55 | 01Jun2012, 01:55 | 0.12 |
| J250 | 0.23 | 8.45 | 01Jun2012, 01:00 | 0.09 |
| J251 | 0.31 | 11.32 | 01Jun2012, 00:55 | 0.10 |
| J260 | 0.19 | 24.46 | 01Jun2012, 01:05 | 0.21 |
| J261 | 0.09 | 5.39 | 01Jun2012, 01:35 | 0.20 |
| J262 | 0.13 | 8.65 | 01Jun2012, 01:20 | 0.16 |

Project: County Hts DBDP Exist Simulation Run: Exist 10 yr

| | | | |
|---|---|---|---|
| Start of Run: | 01Jun2012, 00:00 | Basin Model: | DBDP Exist |
| End of Run: | 02Jun2012, 01:00 | Meteorologic Model: | 10 Year Storm |
| Compute Time: | 20Sep2012, 15:10:35 | Control Specifications: | Control 1 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 19.06 | 01Jun2012, 01:50 | 0.51 |
| 10 | 0.40 | 19.97 | 01Jun2012, 02:15 | 0.58 |
| 11 | 0.40 | 19.97 | 01Jun2012, 02:25 | 0.58 |
| 12 | 0.49 | 78.33 | 01Jun2012, 00:55 | 0.62 |
| 13 | 1.16 | 209.66 | 01Jun2012, 00:55 | 0.62 |
| 14 | 1.16 | 208.43 | 01Jun2012, 01:00 | 0.62 |
| 15 | 1.20 | 234.80 | 01Jun2012, 01:00 | 0.63 |
| 16 | 1.36 | 254.55 | 01Jun2012, 01:10 | 0.65 |
| 17 | 1.45 | 299.67 | 01Jun2012, 01:10 | 0.66 |
| 17A | 1.45 | 295.35 | 01Jun2012, 01:15 | 0.66 |
| 18 | 1.59 | 350.17 | 01Jun2012, 01:20 | 0.67 |
| 19 | 1.59 | 346.54 | 01Jun2012, 01:25 | 0.67 |
| 2 | 0.31 | 19.05 | 01Jun2012, 01:55 | 0.51 |
| 20 | 1.77 | 409.46 | 01Jun2012, 01:25 | 0.68 |
| 21 | 2.08 | 444.28 | 01Jun2012, 01:25 | 0.65 |
| 22 | 2.08 | 436.99 | 01Jun2012, 01:35 | 0.65 |
| 23 | 0.09 | 74.01 | 01Jun2012, 00:55 | 0.83 |
| 3 | 0.39 | 37.98 | 01Jun2012, 01:05 | 0.51 |
| 4 | 0.39 | 37.68 | 01Jun2012, 01:10 | 0.51 |
| 5 | 0.48 | 55.82 | 01Jun2012, 01:25 | 0.55 |
| 50 | 0.17 | 1.07 | 01Jun2012, 04:20 | 0.20 |
| 51 | 0.23 | 11.41 | 01Jun2012, 01:40 | 0.35 |
| 52 | 0.23 | 11.34 | 01Jun2012, 01:55 | 0.35 |
| 53 | 0.31 | 67.03 | 01Jun2012, 01:00 | 0.47 |
| 6 | 0.48 | 55.63 | 01Jun2012, 01:30 | 0.55 |
| 7 | 0.48 | 55.58 | 01Jun2012, 01:35 | 0.55 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 99.07 | 01Jun2012, 01:15 | 0.51 |
| 9 | 0.40 | 19.97 | 01Jun2012, 02:15 | 0.58 |
| Basin 10E | 0.14 | 58.24 | 01Jun2012, 01:15 | 0.81 |
| Basin 11E | 0.18 | 86.37 | 01Jun2012, 01:10 | 0.75 |
| Basin 12E | 0.08 | 67.73 | 01Jun2012, 00:55 | 0.83 |
| Basin 13E | 0.17 | 78.08 | 01Jun2012, 01:15 | 0.90 |
| Basin14E | 0.17 | 71.64 | 01Jun2012, 01:15 | 0.80 |
| Basin 15E | 0.06 | 39.19 | 01Jun2012, 01:00 | 0.80 |
| Basin 16E | 0.19 | 114.85 | 01Jun2012, 01:05 | 0.87 |
| Basin 17E | 0.09 | 27.67 | 01Jun2012, 01:40 | 1.04 |
| Basin 18E | 0.13 | 50.99 | 01Jun2012, 01:20 | 0.95 |
| Basin-1E | 0.31 | 146.72 | 01Jun2012, 01:00 | 0.51 |
| Basin-2E | 0.08 | 62.15 | 01Jun2012, 00:50 | 0.51 |
| Basin 3E | 0.09 | 63.52 | 01Jun2012, 00:55 | 0.72 |
| Basin 4E | 0.19 | 131.08 | 01Jun2012, 00:55 | 0.81 |
| Basin 5E | 0.25 | 100.72 | 01Jun2012, 01:05 | 0.51 |
| Basin 6E | 0.15 | 87.03 | 01Jun2012, 01:00 | 0.69 |
| Basin 7E | 0.09 | 78.51 | 01Jun2012, 00:50 | 0.83 |
| Basin 8East Exist | 0.04 | 29.28 | 01Jun2012, 00:55 | 0.82 |
| Basin 8West Exist | 0.16 | 41.30 | 01Jun2012, 01:30 | 0.78 |
| Basin 9E | 0.09 | 75.92 | 01Jun2012, 00:50 | 0.83 |
| Det Pond 100 Ex | 0.40 | 19.97 | 01Jun2012, 02:15 | 0.58 |
| Det Pond 101 Ex | 0.48 | 55.83 | 01Jun2012, 01:25 | 0.55 |
| Det Pond 102Ex | 0.17 | 1.07 | 01Jun2012, 04:20 | 0.20 |
| Det Pond 103 Ex | 0.31 | 19.06 | 01Jun2012, 01:50 | 0.51 |
| Det Pond 104 Ex | 0.39 | 38.03 | 01Jun2012, 01:05 | 0.51 |
| Det Pond 105 Ex | 0.23 | 11.42 | 01Jun2012, 01:40 | 0.35 |
| J201 | 0.39 | 66.89 | 01Jun2012, 00:50 | 0.51 |
| J202 | 0.48 | 95.53 | 01Jun2012, 00:55 | 0.55 |
| J203 | 0.40 | 167.92 | 01Jun2012, 01:05 | 0.58 |
| J204 | 0.49 | 79.44 | 01Jun2012, 00:50 | 0.62 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J205 | 0.67 | 132.41 | 01Jun2012, 00:55 | 0.62 |
| J206 | 1.16 | 210.74 | 01Jun2012, 00:55 | 0.62 |
| J207 | 1.20 | 236.09 | 01Jun2012, 01:00 | 0.63 |
| J208 | 1.36 | 256.52 | 01Jun2012, 01:00 | 0.65 |
| J209 | 1.45 | 301.67 | 01Jun2012, 01:05 | 0.66 |
| J210 | 1.59 | 353.59 | 01Jun2012, 01:15 | 0.67 |
| J211 | 1.77 | 409.70 | 01Jun2012, 01:25 | 0.68 |
| J212 | 2.08 | 445.70 | 01Jun2012, 01:20 | 0.65 |
| J213 | 2.25 | 504.08 | 01Jun2012, 01:30 | 0.67 |
| J250 | 0.23 | 39.33 | 01Jun2012, 01:00 | 0.36 |
| J251 | 0.31 | 68.02 | 01Jun2012, 00:55 | 0.47 |
| J260 | 0.19 | 114.85 | 01Jun2012, 01:05 | 0.87 |
| J261 | 0.09 | 27.67 | 01Jun2012, 01:40 | 1.04 |
| J262 | 0.13 | 50.99 | 01Jun2012, 01:20 | 0.95 |

Project: County Hts DBDP Exist Simulation Run: Exist 100 Yr

| | | | |
|---|---|---|---|
| Start of Run: | 01Jun2012, 00:00 | Basin Model: | DBDP Exist |
| End of Run: | 02Jun2012, 01:00 | Meteorologic Model: | 100 Year Storm |
| Compute Time: | 20Sep2012, 15:09:54 | Control Specifications: | Control 1 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 68.45 | 01Jun2012, 01:45 | 1.44 |
| 10 | 0.40 | 26.98 | 01Jun2012, 02:35 | 1.52 |
| 11 | 0.40 | 26.98 | 01Jun2012, 02:40 | 1.52 |
| 12 | 0.49 | 165.15 | 01Jun2012, 00:55 | 1.57 |
| 13 | 1.16 | 457.99 | 01Jun2012, 01:00 | 1.57 |
| 14 | 1.16 | 456.77 | 01Jun2012, 01:00 | 1.57 |
| 15 | 1.20 | 514.61 | 01Jun2012, 01:00 | 1.58 |
| 16 | 1.36 | 562.28 | 01Jun2012, 01:10 | 1.60 |
| 17 | 1.45 | 664.80 | 01Jun2012, 01:10 | 1.61 |
| 17A | 1.45 | 660.40 | 01Jun2012, 01:10 | 1.61 |
| 18 | 1.59 | 778.66 | 01Jun2012, 01:15 | 1.62 |
| 19 | 1.59 | 773.19 | 01Jun2012, 01:20 | 1.62 |
| 2 | 0.31 | 68.42 | 01Jun2012, 01:50 | 1.44 |
| 20 | 1.77 | 942.98 | 01Jun2012, 01:20 | 1.64 |
| 21 | 2.08 | 1034.64 | 01Jun2012, 01:20 | 1.55 |
| 22 | 2.08 | 1024.21 | 01Jun2012, 01:25 | 1.55 |
| 23 | 0.09 | 154.67 | 01Jun2012, 00:55 | 1.80 |
| 3 | 0.39 | 62.52 | 01Jun2012, 02:30 | 1.44 |
| 4 | 0.39 | 62.52 | 01Jun2012, 02:30 | 1.44 |
| 5 | 0.48 | 88.69 | 01Jun2012, 01:35 | 1.48 |
| 50 | 0.17 | 1.77 | 01Jun2012, 04:30 | 0.36 |
| 51 | 0.23 | 33.49 | 01Jun2012, 01:30 | 0.71 |
| 52 | 0.23 | 33.08 | 01Jun2012, 01:45 | 0.71 |
| 53 | 0.31 | 140.03 | 01Jun2012, 01:00 | 1.01 |
| 6 | 0.48 | 88.64 | 01Jun2012, 01:45 | 1.48 |
| 7 | 0.48 | 88.60 | 01Jun2012, 01:50 | 1.48 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 268.17 | 01Jun2012, 01:10 | 1.44 |
| 9 | 0.40 | 26.98 | 01Jun2012, 02:30 | 1.52 |
| Basin 10E | 0.14 | 128.37 | 01Jun2012, 01:15 | 1.80 |
| Basin 11E | 0.18 | 196.53 | 01Jun2012, 01:10 | 1.79 |
| Basin 12E | 0.08 | 140.81 | 01Jun2012, 00:55 | 1.88 |
| Basin 13E | 0.17 | 165.60 | 01Jun2012, 01:15 | 1.95 |
| Basin14E | 0.17 | 158.21 | 01Jun2012, 01:15 | 1.77 |
| Basin 15E | 0.06 | 84.66 | 01Jun2012, 01:00 | 1.77 |
| Basin 16E | 0.19 | 244.24 | 01Jun2012, 01:05 | 1.89 |
| Basin 17E | 0.09 | 55.58 | 01Jun2012, 01:35 | 2.11 |
| Basin 18E | 0.13 | 105.55 | 01Jun2012, 01:20 | 2.01 |
| Basin-1E | 0.31 | 386.93 | 01Jun2012, 01:00 | 1.44 |
| Basin-2E | 0.08 | 154.00 | 01Jun2012, 00:50 | 1.44 |
| Basin 3E | 0.09 | 141.26 | 01Jun2012, 00:55 | 1.68 |
| Basin 4E | 0.19 | 280.20 | 01Jun2012, 00:55 | 1.78 |
| Basin 5E | 0.25 | 270.37 | 01Jun2012, 01:05 | 1.43 |
| Basin 6E | 0.15 | 199.82 | 01Jun2012, 01:00 | 1.65 |
| Basin 7E | 0.09 | 163.07 | 01Jun2012, 00:50 | 1.81 |
| Basin 8East Exist | 0.04 | 62.18 | 01Jun2012, 00:55 | 1.79 |
| Basin 8West Exist | 0.16 | 92.95 | 01Jun2012, 01:30 | 1.75 |
| Basin 9E | 0.09 | 158.22 | 01Jun2012, 00:50 | 1.80 |
| Det Pond 100 Ex | 0.40 | 26.98 | 01Jun2012, 02:30 | 1.52 |
| Det Pond 101 Ex | 0.48 | 88.70 | 01Jun2012, 01:35 | 1.48 |
| Det Pond 102Ex | 0.17 | 1.77 | 01Jun2012, 04:30 | 0.36 |
| Det Pond 103 Ex | 0.31 | 68.46 | 01Jun2012, 01:45 | 1.44 |
| Det Pond 104 Ex | 0.39 | 62.53 | 01Jun2012, 02:25 | 1.44 |
| Det Pond 105 Ex | 0.23 | 33.59 | 01Jun2012, 01:30 | 0.71 |
| J201 | 0.39 | 164.70 | 01Jun2012, 00:50 | 1.44 |
| J202 | 0.48 | 190.42 | 01Jun2012, 00:55 | 1.48 |
| J203 | 0.40 | 435.22 | 01Jun2012, 01:05 | 1.52 |
| J204 | 0.49 | 166.99 | 01Jun2012, 00:50 | 1.57 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J205 | 0.67 | 302.08 | 01Jun2012, 01:00 | 1.57 |
| J206 | 1.16 | 458.17 | 01Jun2012, 00:55 | 1.57 |
| J207 | 1.20 | 516.12 | 01Jun2012, 01:00 | 1.58 |
| J208 | 1.36 | 565.26 | 01Jun2012, 01:05 | 1.60 |
| J209 | 1.45 | 674.96 | 01Jun2012, 01:05 | 1.61 |
| J210 | 1.59 | 784.16 | 01Jun2012, 01:10 | 1.62 |
| J211 | 1.77 | 943.69 | 01Jun2012, 01:20 | 1.64 |
| J212 | 2.08 | 1035.10 | 01Jun2012, 01:20 | 1.55 |
| J213 | 2.25 | 1181.78 | 01Jun2012, 01:25 | 1.58 |
| J250 | 0.23 | 84.99 | 01Jun2012, 01:00 | 0.73 |
| J251 | 0.31 | 141.69 | 01Jun2012, 00:55 | 1.01 |
| J260 | 0.19 | 244.24 | 01Jun2012, 01:05 | 1.89 |
| J261 | 0.09 | 55.58 | 01Jun2012, 01:35 | 2.11 |
| J262 | 0.13 | 105.55 | 01Jun2012, 01:20 | 2.01 |

# APPENDIX B

DATA AND PRINTOUTS

FOR

**FUTURE LAND USE**
**AND**
**FUTURE (DBDPA) HYDRAULIC CONDITIONS**

The following input data tables are included in Appendix A:

- B-1 SUMMARY OF BASIN INPUT DATA FUTURE LAND USE CONDITIONS
- B-2 GREEN AND AMPT LOSS DATA EXISTING AND FUTURE LAND USE CONDITIONS
- B-3 FUTURE LAND USE IMPERVIOUSNESS
- B-4 LAG TIME FOR FUTURE CONDITIONS
- DETENTION POND STAGE STORAGE DISCHARGE CURVES

The following direct printouts from HMS are included in Appendix A:

- 2 Year HMS Summary Output Table
- 10 Year HMS Summary Output Table
- 100 Year HMS Summary Output Table

***Users of this report need to be aware that the HMS program routes only flows entering the upstream end of the element and ignores the possibility that any adjacent sub-basin flow may be entering the element. Due to this limitation the user must exercise caution when using Model calculated peak channel and pipe flows. Flows for design purposes must be increased appropriately using engineering judgement or other suitable method to account for incoming sub-basin flows.***

*(Note: HMS Printout date of September 2012 is correct as this was that date the original review submittal for the report was made to the City. Multiyear review period ensured and HMS model was not rerun between that date and the time of this Final Report preparation)*

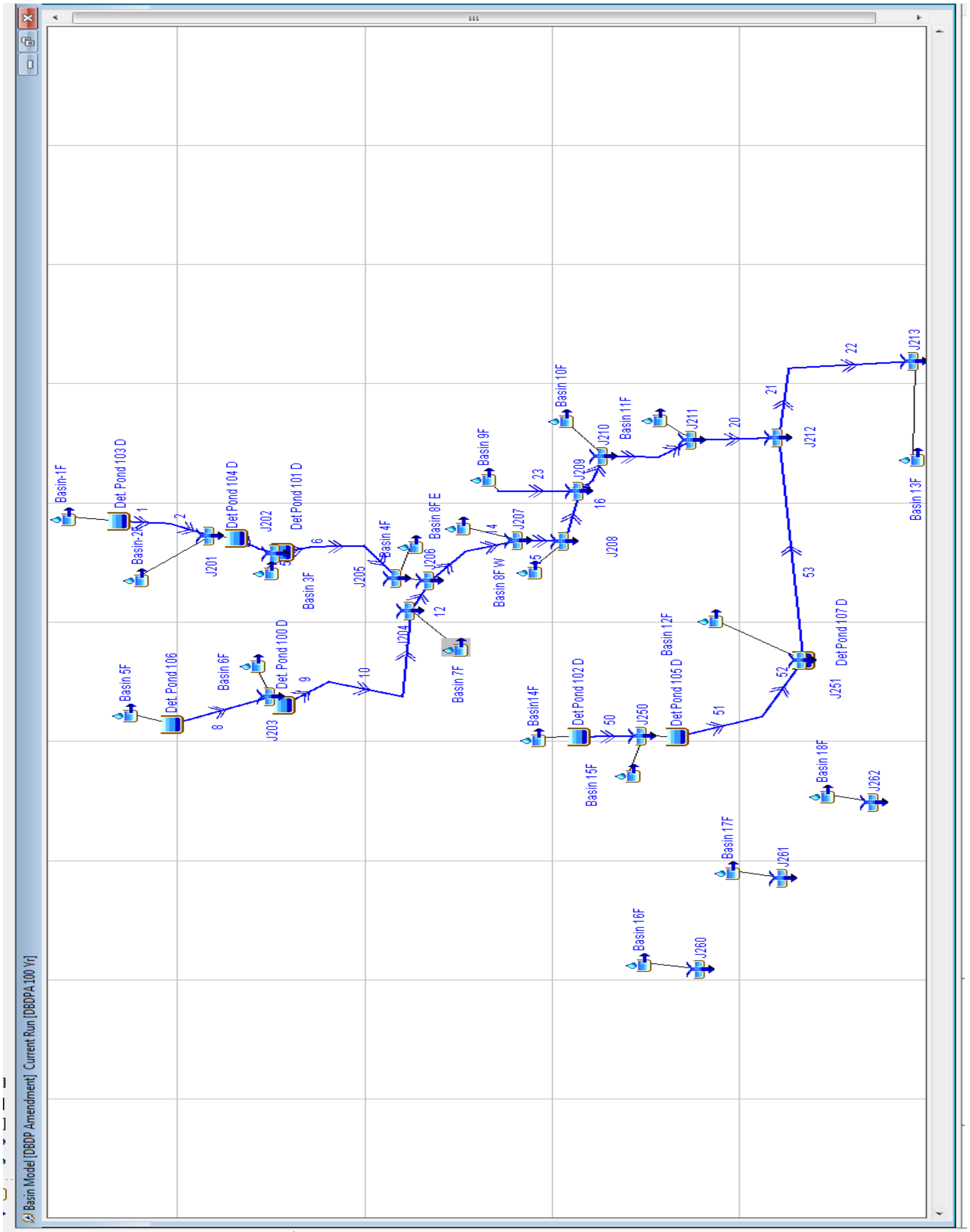

Basin Model [DBDP Amendment] Current Run [DBDPA 100 Yr]
Basin-1F
Det Pond 103 D
Basin-2F
J201
Det Pond 104 D
J202
Det Pond 101 D
Basin 3F
J205
Basin 4F
J206
Basin 8F E
Basin 8F W
J207
J208
Basin 9F
J209
Basin 10F
J210
Basin 11F
J211
J212
Basin 13F
J213
J204
Basin 7F
Basin 5F
Det. Pond 106
Basin 6F
Det. Pond 100 D
J203
Basin14F
Det Pond 102 D
Basin 15F
J250
Det Pond 105 D
Basin 12F
J251
Det Pond 107 D
Basin 18F
J262
Basin 17F
J261
Basin 16F
J260

TABLE B-1

COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

SUMMARY OF BASIN INPUT DATA

FUTURE LAND USE CONDITIONS

| SUBBASIN NUMBER | AREA (SQ MI) | AREA (ACRES) | INITIAL CONTENT | SATURATED CONTENT | SUCTION (IN) | CONDUCTIVITY (IN/HR) | IMPERVIOUS (EIA) % | INITIAL STORAGE % | MAX STORAGE (IN) | LAG TIME (HR) | PEAKING COEF |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Basin-1F | 0.31 | 198 | 0.116 | 0.434 | 3.500 | 0.512 | 30.3 | 0 | 0.35 | 0.33 | 0.7 |
| Basin-2F | 0.08 | 51 | 0.116 | 0.434 | 3.500 | 0.512 | 41.0 | 0 | 0.35 | 0.17 | 0.7 |
| Basin 3F | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 37.8 | 0 | 0.35 | 0.29 | 0.6 |
| Basin 4F | 0.19 | 122 | 0.116 | 0.434 | 3.500 | 0.512 | 25.6 | 0 | 0.35 | 0.38 | 0.6 |
| Basin 5F | 0.25 | 160 | 0.116 | 0.434 | 3.539 | 0.510 | 41.6 | 0 | 0.35 | 0.37 | 0.7 |
| Basin 6F | 0.15 | 96 | 0.116 | 0.434 | 3.500 | 0.512 | 48.3 | 0 | 0.35 | 0.31 | 0.6 |
| Basin 7F | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 27.8 | 0 | 0.35 | 0.29 | 0.6 |
| Basin 8F E | 0.04 | 26 | 0.116 | 0.434 | 3.500 | 0.512 | 33.2 | 0 | 0.35 | 0.33 | 0.6 |
| Basin 8F W | 0.16 | 102 | 0.116 | 0.434 | 3.500 | 0.512 | 25.9 | 0 | 0.35 | 0.91 | 0.5 |
| Basin 9F | 0.09 | 58 | 0.116 | 0.434 | 3.500 | 0.512 | 30.3 | 0 | 0.35 | 0.29 | 0.6 |
| Basin 10F | 0.14 | 90 | 0.116 | 0.364 | 3.500 | 0.512 | 25.7 | 0 | 0.35 | 0.69 | 0.6 |
| Basin 11F | 0.18 | 115 | 0.138 | 0.346 | 6.219 | 0.264 | 31.6 | 0 | 0.35 | 0.46 | 0.6 |
| Basin 12F | 0.08 | 51 | 0.144 | 0.42 | 6.837 | 0.205 | 61.6 | 0 | 0.35 | 0.21 | 0.6 |
| Basin 13F | 0.17 | 109 | 0.169 | 0.434 | 10.459 | 0.081 | 19.6 | 0 | 0.4 | 0.63 | 0.6 |
| Basin14F | 0.17 | 109 | 0.116 | 0.434 | 3.500 | 0.512 | 28.4 | 0 | 0.35 | 0.66 | 0.6 |
| Basin 15F | 0.06 | 38 | 0.116 | 0.434 | 3.500 | 0.512 | 34.3 | 0 | 0.35 | 0.37 | 0.6 |
| Basin 16F | 0.19 | 122 | 0.133 | 0.399 | 5.610 | 0.325 | 40.7 | 0 | 0.35 | 0.44 | 0.6 |
| Basin 17F | 0.09 | 58 | 0.158 | 0.331 | 8.664 | 0.079 | 28.4 | 0 | 0.4 | 0.86 | 0.5 |
| Basin 18F | 0.13 | 83 | 0.157 | 0.35 | 8.613 | 0.105 | 26.2 | 0 | 0.4 | 0.61 | 0.5 |

TABLE B2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 1

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 15.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 80.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ZnD | Loam | 105.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 201 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

0.116

SUBBASIN 2

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 3.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 15.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 0.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 28.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 48 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 = fc/2 = 0.116

SUBBASIN 3

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 30.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 12.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 1.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 13.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 59 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 = 0.116

SUBBASIN 4

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydrologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 2.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 40.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 34 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| ZnD | Loam | 7.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 36.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 121 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |

fc/2 = 0.116

TABLE B2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 5

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 11.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| NuC | Loam | 41.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 41.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ZnD | Loam | 64.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| ScE | Clay | 0.7 | 0.012 | 0.024 | 12.453 | 0.475 | 0.385 | 0.378 | 0.196 | D |
| Weighted Average for Subbasin | | 160 | 0.255 | 0.510 | 3.539 | | 0.434 | 0.233 | 0.317 | |
| | | | | | | | fc/2 = | 0.116 | | |

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SUBBASIN 6

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 38.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| NdE | Loam(1) | 30.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 3.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 27.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 99 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 7

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| NuB | Loam | 7.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| Nde | Loam(1) | 14.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 21.9 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 10.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 55 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | | 0.116 | | |

SUBBASIN 8E

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Nde | Loam(1) | 1.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 18.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 6.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 26 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

TABLE B2
COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 8W

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SkA | Loam(3) | 102 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 0.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 102 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 9

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Nde | Loam(1) | 5.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| NwA | Loam | 0.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | C |
| SkA | Loam(3) | 29.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam (3) | 21.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 58 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 10

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 8.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 55.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 18.6 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 6.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 89 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 11

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Eg | Silty Clay Loam | 1 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| SeA | Loam | 11.3 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 5.7 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay Loam | 63.1 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 31.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 112 | 0.132 | 0.264 | 6.219 | | 0.364 | 0.277 | 0.225 | |
| | | | | | | | fc/2 = | 0.138 | | |

TABLE B2
COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 12

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SkB | Loam(2) | 3.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay Loam | 34.5 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 11.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 49 | 0.103 | 0.205 | 6.837 | | 0.346 | 0.287 | 0.202 | |
| | | | | | | | fc/2 = | 0.144 | | |

SUBBASIN 13

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Bp | Silt Loam | 0.5 | 0.134 | 0.268 | 6.567 | 0.501 | 0.486 | 0.284 | 0.344 | B |
| Co | Loam | 0.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 96 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 10.3 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 107 | 0.040 | 0.081 | 10.459 | | 0.420 | 0.338 | 0.251 | |
| | | | | | | | fc/2 = | 0.169 | | |

SUBBASIN 14

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 13.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 49.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 43.2 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 106 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

SUBBASIN 15

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| SeA | Loam | 6.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 23.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Znd | Loam | 6.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 36 | 0.256 | 0.512 | 3.500 | | 0.434 | 0.232 | 0.318 | |
| | | | | | | | fc/2 = | 0.116 | | |

TABLE B2

COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT

GREEN AND AMPT LOSS PARAMETERS (SAME DATA FOR EXISTING AND FUTURE LAND USE)

SUBBASIN 16

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Eg | Silty Clay Loam | 0.9 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| EnD | Silt Loam | 18.1 | 0.134 | 0.268 | 6.567 | 0.501 | 0.486 | 0.284 | 0.344 | D |
| SeA | Loam | 45.5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkA | Loam(3) | 4.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| SkB | Loam(2) | 0.8 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Ow | Clay loam | 41.3 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Znd | Loam | 10.4 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Weighted Average for Subbasin | | 122 | 0.163 | 0.325 | 5.610 | | 0.399 | 0.267 | 0.266 | |
| | | | | | | | fc/2 = | 0.133 | | |

SUBBASIN 17

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Co | Loam | 0.1 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 9.8 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 44.8 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 55 | 0.039 | 0.079 | 8.664 | | 0.331 | 0.316 | 0.173 | |
| | | | | | | | fc/2 = | 0.158 | | |

SUBBASIN 18

| Soil Symbol | Texture | Area acres | Bare Soil* Hyd Cond. K (in/hr)* | Vegetated** Hyd. Cond. K (in/hr) | Suction Head S (in) | Porosity | Effective Porosity ef | Field Capacity fc | Moisture Deficit ef - 0.5fc | NRCS Hydologic Soil Type |
|---|---|---|---|---|---|---|---|---|---|---|
| Co | Loam | 5 | 0.256 | 0.512 | 3.500 | 0.463 | 0.434 | 0.232 | 0.318 | B |
| Eg | Silty Clay Loam | 21.7 | 0.039 | 0.078 | 10.748 | 0.471 | 0.432 | 0.342 | 0.261 | D |
| Ow | Clay loam | 52.8 | 0.039 | 0.078 | 8.220 | 0.464 | 0.309 | 0.31 | 0.154 | C |
| Weighted Average for Subbasin | | 80 | 0.053 | 0.105 | 8.613 | | 0.350 | 0.314 | 0.194 | |
| | | | | | | | fc/2 = | 0.157 | | |

(1) NdE is Gravelly Loam - Assume as Loam

(2) SkB is Santanta Urban Land Comples - Assumed as Loam and as Type B

(3) SkA is Santanta Urban Land Comples - Assumed as Loam and as Type B

* Bare soil hydraulic conductivity values from City of Rapid City Drainage Manual

** Vegetated soil hydraulic conductivity values assumed at 2 times bare soil.

TABLE B-3 COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
FUTURE BASIN IMPERVIOUSNESS

| BASIN #1F | NC | MDR | LDR/PRD | LDR | Open | OFFICE | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1.5 U/AC | | Space | w/PCD | | AREA | % | % | % |
| % Impervious | 70 | 60 | 27 | 36 | 2 | 95 | 50 | | | | |
| Acres | 14.6 | 21.8 | 135.3 | 8.3 | 4 | 5.7 | 11.6 | 201 | 36.8 | 22.3 | 30.3 |

| BASIN #2F | | | | | Open | OFFICE | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | LDR | Space | w/PCD | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 36 | 2 | 95 | 50 | | | | |
| Acres | 0 | 0 | 0 | 34.3 | 1.5 | 9.5 | 2.7 | 48 | 47.4 | 32.6 | 41.0 |

| BASIN #3F | | | | | Open | | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | LDR | Space | MDR | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 34 | 2 | 60 | 50 | | | | |
| Acres | 0 | 0 | 0 | 18.7 | 2.1 | 6.5 | 31.5 | 59 | 44.3 | 29.5 | 37.8 |

| BASIN #4F | XX | XX | XX | XX | XX | LDR | XX | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 32 | 0 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 120.5 | 0 | 121 | 32.0 | 18.1 | 25.6 |

| BASIN #5F | GC | MDR | LDR/PRD | LI | LDR/PRD | OFFICE | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1.5 U/AC | | | w/PCD | | AREA | % | % | % |
| % Impervious | 95 | 60 | 27 | 80 | 33 | 95 | 50 | | | | |
| Acres | 2.5 | 17.4 | 93.2 | 1.1 | 3.4 | 34.7 | 7.8 | 160 | 48.0 | 33.3 | 41.6 |

| BASIN #6F | XX | MDR | GC | LDR | Open | OFFICE | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | w/PCD | | AREA | % | % | % |
| % Imp | 0 | 60 | 95 | 36 | 2 | 95 | 50 | | | | |
| Acres | 0 | 41.3 | 3.2 | 24.7 | 5.9 | 11.2 | 13.1 | 99 | 54.3 | 40.1 | 48.3 |

TABLE B-3  COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
FUTURE BASIN IMPERVIOUSNESS

| BASIN #7F | XX | XX | XX | XX | Open | LDR | MDR | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Space | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 2 | 35 | 60 | | | | |
| Acres | 0 | 0 | 0 | 0 | 2.5 | 50.6 | 1.7 | 55 | 34.3 | 20.1 | 27.8 |

| BASIN #8F East | XX | XX | XX | LDR | PUD | Open | MHP | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Space | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 31 | 48 | 2 | 35 | | | | |
| Acres | 0 | 0 | 0 | 6.4 | 16.6 | 2 | 1 | 26 | 39.8 | 25.1 | 33.2 |

| BASIN #8F West | XX | XX | XX | XX | XX | LDR | MHP | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 31 | 35 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 68 | 34 | 102 | 32.3 | 18.4 | 25.9 |

| BASIN #9F | XX | XX | XX | XX | XX | LDR | PUD | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 32 | 48 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 40.2 | 17.4 | 58 | 36.8 | 22.4 | 30.3 |

| BASIN #10F | XX | XX | XX | XX | LI | LDR | PUD | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 80 | 30 | 35 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0.8 | 58.3 | 30.3 | 89 | 32.1 | 18.2 | 25.7 |

| BASIN #11F | XX | XX | XX | HWY 44 | MDR | LDR | PUBLIC | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | ROW | | | FIRE STA | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 32 | 60 | 32 | 80 | | | | |
| Acres | 0 | 0 | 0 | 14.5 | 12.9 | 77.9 | 6.9 | 112 | 38.2 | 23.6 | 31.6 |

TABLE B-3 COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
FUTURE BASIN IMPERVIOUSNESS

| BASIN #12F | XX | XX | HWY 44 | NC | MDR | OFFICE | LI | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | ROW | | | w/PCD | | AREA | % | % | % |
| % Impervious | 0 | 0 | 32 | 70 | 60 | 95 | 80 | | | | |
| Acres | 0 | 0 | 11 | 4.5 | 16.2 | 15.9 | 1.2 | 49 | 66.5 | 54.2 | 61.6 |

| BASIN #13F | | | | | | | PUBLIC OR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | LDR | NC | HWY 44 | VERY LOW | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | ROW | LDR | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 36 | 70 | 2 | 7 | | | | |
| Acres | 0 | 0 | 0 | 57.9 | 5.4 | 5.8 | 38 | 107 | 25.6 | 12.9 | 19.6 |

| BASIN #14F | | | | | | | PUBLIC OR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | PUD | LDR | VERY LOW | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | | LDR | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 36 | 34 | 7 | | | | |
| Acres | 0 | 0 | 0 | 0 | 45.1 | 60.4 | 0 | 106 | 34.9 | 20.6 | 28.4 |

| BASIN #15F | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | PUD | LI | MDR | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | | | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 34 | 80 | 60 | | | | |
| Acres | 0 | 0 | 0 | 0 | 27.5 | 0.9 | 8 | 36 | 40.9 | 26.1 | 34.3 |

| BASIN #16F | GC | Hwy 44 | MHP | LDR | PUD | OFFICE | SCHOOL | TOTAL | MIA | EIA AVE | EIA HD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ROW | | | | w/PCD | | AREA | % | % | % |
| % Impervious | 95 | 33 | 40 | 35 | 51 | 95 | 50 | | | | |
| Acres | 1.5 | 16.8 | 0.5 | 32.2 | 48.3 | 6.8 | 15.7 | 122 | 47.1 | 32.3 | 40.7 |

| BASIN #17F | | | | | | | PUBLIC OR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | XX | XX | LDR | VERY LOW | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | | | LDR | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 0 | 0 | 36 | 7 | | | | |
| Acres | 0 | 0 | 0 | 0 | 0 | 52.7 | 2 | 55 | 34.9 | 20.7 | 28.4 |

TABLE B-3  COUNTY HEIGHTS DRAINAGE BASIN PLAN AMENDMENT
FUTURE BASIN IMPERVIOUSNESS

| BASIN #18F | | | | | | | PUBLIC OR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | XX | XX | XX | NC | Hwy 44 | LDR | VERY LOW | TOTAL | MIA | EIA AVE | EIA HD |
| | | | | | ROW | | LDR | AREA | % | % | % |
| % Impervious | 0 | 0 | 0 | 70 | 2 | 36 | 7 | | | | |
| Acres | 0 | 0 | 0 | 6.4 | 3.1 | 57 | 13 | 80 | 32.7 | 18.7 | 26.2 |

XX = This column not used

LI = Light Industrial

GC = General Commercial

NC = Neighborhood Commerical

MDR = Multi Unit (Impervious = Average of detached and attached)

LDR/PRD 1.5 U/Ac = Low Density Residential with Planned Development,  Density of 1.5 unit per acre

LDR = Low Density Residential, Imperviouse based Curve Developed within study area by FMG
Existing Units/Acre Measured From Aerials, Future Developed Assumes 3.4 units/acre

OFFICE WITH PCD = Office Commerial with Planned Development

SCHOOL = Public School

MHP = Mobile Home Park, Impervious assumed at 40% based on estimates of existing parks

PUD = Planned Unit Development, Imperviouse based on combination existing and proposed uses in PUD

HWY 44 ROW = Highway 44 right of way, including railroad right of way, imperviouse based on existing pavements

PUBLIC FIRE STA = Public Land used for fire station, impervious assumed same as light industrial

PUBLIC OR VERY LOW LDR = Impervious based on parks/cemeteries or 0.5 housing units/acre

OPEN SPACE = Open space/undeveloped at Existing Ponds 100, 101, 103, and 104

TOTAL AREA = Total size of drainage basin

MIA = Mapped Imperviouse Area in Percent

EIA  AVE = Effective Imperviouse Area in percent based on Average Sutherland Equation (Included in Table for comparision only

**EIA  HD  = Effective Imperviouse Area in percent based on Highly Connected Sutherland Equation (This equation used in study for EIA)**

TABLE B-4 COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT

TABLE OF INFORMATION FOR DETERMINING LAG TIME - FUTURE LAND USE DBDP CONDITIONS

| Basin # | Area (Sq Mi) | Area (Ac) | Future Mapped Imperv (%) | I (ft) | L (Mi) | lc (ft) | Lc (Mi) | A Soils CN 39 (Ac) | B Soils CN 61 (Ac) | C Soils CN 74 (Ac) | D Soils CN 80 (Ac) | Total Area (Ac) | Ave CN For Grass | Overall Ave CN Including MIA | S Max Pot Retention (in) | 10' Contour Lengths (ft) | Y Average and Slop (%) | **Calculated NRCS Lag Time Used In Report (Hours)** | Comparison Lag Time From Snyder Equation Using Ct = .6 (Hours) | Comparison Snyder Ct Coef. Back Calculated From NRCS Lag Time* | Comparison Lag Time Calculated With Snyder Eq. and Ct from USACE Method (Hours) | Comparison Snyder Ct Coef Calculated From USACE Equation** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.31 | 201 | 36.8 | 3855 | 0.73 | 1907 | 0.36 | 0 | 185 | 16 | 0 | 201 | 62.0 | 75.3 | 3.288 | 91178 | 10.41 | **0.33** | 0.40 | 0.50 | 0.31 | 0.47 |
| 2 | 0.08 | 48 | 47.4 | 1849 | 0.35 | 637 | 0.12 | 0 | 44 | 4 | 0 | 48 | 62.1 | 79.1 | 2.636 | 20771 | 9.93 | **0.17** | 0.23 | 0.44 | 0.15 | 0.39 |
| 3 | 0.09 | 59 | 44.3 | 2973 | 0.56 | 1057 | 0.20 | 0 | 27 | 32 | 0 | 59 | 68.1 | 81.3 | 2.293 | 16643 | 6.48 | **0.29** | 0.31 | 0.55 | 0.21 | 0.41 |
| 4 | 0.19 | 121 | 32.0 | 3798 | 0.72 | 1537 | 0.29 | 0 | 85 | 36 | 0 | 121 | 64.9 | 75.5 | 3.250 | 39944 | 7.58 | **0.38** | 0.38 | 0.61 | 0.33 | 0.52 |
| 5 | 0.25 | 160 | 48.0 | 4662 | 0.88 | 2191 | 0.41 | 0 | 106 | 53 | 1 | 160 | 65.4 | 81.1 | 2.337 | 56975 | 8.17 | **0.37** | 0.44 | 0.50 | 0.28 | 0.38 |
| 6 | 0.15 | 99 | 54.3 | 3664 | 0.69 | 1629 | 0.31 | 0 | 60 | 39 | 0 | 99 | 66.1 | 83.4 | 1.988 | 29887 | 6.93 | **0.31** | 0.38 | 0.48 | 0.22 | 0.35 |
| 7 | 0.09 | 55 | 34.3 | 2669 | 0.51 | 1268 | 0.24 | 0 | 47 | 8 | 0 | 55 | 62.8 | 74.9 | 3.356 | 18501 | 7.72 | **0.29** | 0.32 | 0.55 | 0.26 | 0.50 |
| 8E | 0.04 | 26 | 39.8 | 3100 | 0.59 | 1139 | 0.22 | 0 | 26 | 0 | 0 | 26 | 61.0 | 75.7 | 3.206 | 8183 | 7.23 | **0.33** | 0.32 | 0.62 | 0.24 | 0.44 |
| 8W | 0.16 | 102 | 32.3 | 4770 | 0.90 | 1982 | 0.38 | 0 | 102 | 0 | 0 | 102 | 61.0 | 73.0 | 3.708 | 9984 | 2.25 | **0.91** | 0.43 | 1.26 | 0.38 | 0.52 |
| 9 | 0.09 | 58 | 36.8 | 2284 | 0.43 | 978 | 0.19 | 0 | 57 | 1 | 0 | 58 | 61.2 | 74.7 | 3.382 | 15265 | 6.04 | **0.29** | 0.28 | 0.62 | 0.22 | 0.47 |
| 10 | 0.14 | 89 | 32.1 | 4285 | 0.81 | 1769 | 0.34 | 0 | 89 | 0 | 0 | 89 | 61.0 | 72.9 | 3.722 | 12898 | 3.33 | **0.69** | 0.41 | 1.02 | 0.35 | 0.52 |
| 11 | 0.18 | 112 | 38.2 | 4343 | 0.82 | 2500 | 0.47 | 0 | 48 | 63 | 1 | 112 | 68.5 | 79.8 | 2.537 | 25247 | 5.17 | **0.46** | 0.45 | 0.60 | 0.34 | 0.46 |
| 12 | 0.08 | 49 | 66.5 | 2760 | 0.52 | 1402 | 0.27 | 0 | 14 | 35 | 0 | 49 | 70.3 | 88.7 | 1.272 | 13386 | 6.27 | **0.21** | 0.33 | 0.38 | 0.16 | 0.30 |
| 13 | 0.17 | 107 | 25.6 | 5047 | 0.96 | 2852 | 0.54 | 0 | 1 | 10 | 96 | 107 | 79.3 | 84.1 | 1.896 | 12166 | 2.61 | **0.63** | 0.49 | 0.77 | 0.51 | 0.62 |
| 14 | 0.17 | 106 | 34.9 | 4501 | 0.85 | 2215 | 0.42 | 0 | 106 | 0 | 0 | 106 | 61.0 | 73.9 | 3.529 | 16883 | 3.66 | **0.66** | 0.44 | 0.90 | 0.36 | 0.49 |
| 15 | 0.06 | 36 | 40.9 | 2755 | 0.52 | 1447 | 0.27 | 0 | 36 | 0 | 0 | 36 | 61.0 | 76.1 | 3.135 | 7464 | 4.76 | **0.37** | 0.33 | 0.66 | 0.24 | 0.43 |
| 16 | 0.19 | 122 | 47.1 | 4512 | 0.85 | 1648 | 0.31 | 0 | 62 | 41 | 19 | 122 | 68.4 | 82.3 | 2.148 | 26631 | 5.01 | **0.44** | 0.40 | 0.65 | 0.26 | 0.39 |
| 17 | 0.09 | 55 | 34.9 | 3415 | 0.65 | 1711 | 0.32 | 0 | 0 | 45 | 10 | 55 | 75.1 | 83.1 | 2.039 | 1928 | 0.80 | **0.86** | 0.38 | 1.37 | 0.31 | 0.49 |
| 18 | 0.13 | 80 | 32.7 | 3820 | 0.72 | 2058 | 0.39 | 0 | 5 | 53 | 22 | 80 | 74.8 | 82.4 | 2.136 | 6958 | 2.00 | **0.61** | 0.41 | 0.89 | 0.35 | 0.51 |

*Note: Ct back calculation is described as dividing the calculated NRCS Lag Time by the Synder Equation to find Ct

**USACE Equation is Ct = 7.81/(i^0.78) where I is imperviousness

Synder Equation is per RCIDCM: tp = Lag Time = Ct(LLc)^0.3

**DETENTION POND 100**

**ELEVATION - STORAGE - DISCHARGE DATA**

**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

**Improved Outlet System and Graded Pool Below 3217 for WQCV**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | WQCV 6" Dia Orf Fl 3210 (CFS) | 12" Dia Orf FL 3213 CFS | 24" Dia Orf FL 3416 CFS | 24" Dia Orf FL 3417 CFS | 48" Dia Riser Weir Rim 3220 (CFS) | 48" Dia Riser Orf Rim 3220 (CFS) | Total Structure Flow (CFS) | Exist 36"" Dia RCP Outlet Cap. (CFS) | Final Riser Discharge Curve (CFS) | 60' Spillway 3222.5 (CFS) | **Overall Final Discharge Curve (CFS)** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3209 | 0 | 0 | **0.0** | | | | | | | 0 | 20 | 0 | | **0.0** |
| 3210 | 4280 | 0.05 | **0.0** | 0.8 | | | | | | 0.8 | 32 | 0.8 | | **0.8** |
| 3211 | 20900 | 0.29 | **0.3** | 1.2 | | | | | | 1.2 | 47 | 1.2 | | **1.2** |
| 3212 | 50650 | 0.82 | **1.2** | 1.5 | | | | | | 1.5 | 55 | 1.5 | | **1.5** |
| 3213 | 86400 | 1.57 | **2.7** | 1.8 | 0.0 | | | | | 1.8 | 62 | 1.8 | | **1.8** |
| 3214 | 112900 | 2.29 | **5.0** | 2.1 | 2.0 | | | | | 4.1 | 68 | 4.1 | | **4.1** |
| 3215 | 134500 | 2.84 | **7.9** | 2.3 | 5.0 | 0 | 0 | | | 7.3 | 74 | 7.3 | | **7.3** |
| 3216 | 141290 | 3.17 | **11.0** | 2.5 | 6.0 | 0 | 0 | | | 8.5 | 80 | 9 | | **8.5** |
| 3217 | 180665 | 3.70 | **14.7** | 2.6 | 7.0 | 8 | 0 | | | 17.6 | 85 | 18 | | **17.6** |
| 3218 | 191097 | 4.27 | **19.0** | 2.7 | 8.0 | 15 | 8 | | | 33.7 | 90 | 34 | | **33.7** |
| 3219 | 199498 | 4.48 | **23.5** | 2.9 | 9.0 | 21 | 15 | | | 47.9 | 95 | 48 | | **47.9** |
| 3220 | 207842 | 4.68 | **28.1** | 3.1 | 9.8 | 26 | 21 | 0 | 0 | 59.9 | 99 | 60 | | **59.9** |
| 3221 | 215938 | 4.86 | **33.0** | 3.2 | 10.5 | 30 | 26 | 41 | ~~61~~ | 110.7 | 104 | 104 | | **104.0** |
| 3222 | 226740 | 5.08 | **38.1** | 3.3 | 11.0 | 34 | 30 | ~~117~~ | 85 | 163.3 | 108 | 108 | | **108.0** |
| 3222.5 | 230000 | 2.62 | **40.7** | 3.4 | 11.5 | 35 | 32 | ~~163~~ | 95 | 176.9 | 110 | 110 | 0 | **110.0** |
| 3223 | 234906 | 2.67 | **43.4** | 3.5 | 11.8 | 37 | 34 | ~~215~~ | 104 | 190.3 | 112 | 112 | 55 | **167.0** |
| 3224 | 243972 | 5.50 | **48.9** | 3.6 | 12.0 | 39 | 37 | ~~331~~ | 121 | 212.6 | 116 | 116 | 335 | **451.0** |

**DETENTION POND 101**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**
**Improved Outlet System, No Pool Grading**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | 12" ORF FL 3221.7 (CFS) | 12" ORF FL 3225 (CFS) | 72" Riser As Weir Top 3227 (CFS) | 72" Riser As Orifice Top 3227 (CFS) | Groove End 36" RCP 3221.2 (CFS) | Final Riser Discharge Curve (CFS) | Raised 25' Spillway 3231 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3221.2 | 0 | 0 | **0** | 0.0 | 0.0 | | | 0 | 0 | | **0** |
| 3222 | 11042 | 0.10 | **0.10** | 2.0 | 0.0 | | | 5 | 2 | | **2** |
| 3223 | 21834 | 0.38 | **0.48** | 4.0 | 0.0 | | | 20 | 4 | | **4** |
| 3224 | 29895 | 0.59 | **1.07** | 6.0 | 0.0 | | | 33 | 6 | | **6** |
| 3225 | 32960 | 0.72 | **1.79** | 7.0 | 0.0 | | | 50 | 7 | | **7** |
| 3226 | 36037 | 0.79 | **2.59** | 8.0 | 3.0 | | | 65 | 11 | | **11** |
| 3227 | 39242 | 0.86 | **3.45** | 9.0 | 5.0 | 0 | 0 | 75 | 14 | | **14** |
| 3228 | 42494 | 0.94 | **4.39** | 9.5 | 6.0 | 62 | ~~136~~ | 85 | 78 | | **78** |
| 3229 | 45968 | 1.02 | **5.40** | 10.0 | 7.0 | 175 | ~~192~~ | 92 | 92 | | **92** |
| 3230 | 49421 | 1.09 | **6.50** | 11.0 | 8.0 | ~~322~~ | 236 | 102 | 102 | 0 | **102** |
| 3231 | 53015 | 1.18 | **7.67** | 11.5 | 9.0 | ~~496~~ | 272 | 110 | 11 | 0 | **110** |
| 3232 | 56636 | 1.26 | **8.93** | 12.0 | 9.5 | ~~693~~ | 304 | 116 | 116 | 66 | **182** |
| 3232.5 | 59346 | 0.67 | **9.60** | 12.5 | 10.0 | ~~800~~ | 320 | 120 | 120 | 120 | **230** |
| 3233 | | | **10.3** | 12.5 | 10.5 | | | 122 | 122 | 174 | **296** |

Contour Area digitized from original design plans for Pond 101

**DETENTION POND 102**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**
**IMPROVED OUTLET SYSTEM AND RESHAPED TOP OF DAM**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | 6" Orifice 3113.8 (CFS) | 30" Dia Riser Weir Rim 3116 (CFS) | 30" Dia Riser Orf Rim 3116 (CFS) | 240' Spillway Elev 3118 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|---|---|
| 3113.8 | 148100 | 0 | **0** | 0.0 | ~~0~~ | 0 | | **0.0** |
| 3114 | 156820 | 0.70 | **0.70** | 0.1 | ~~0~~ | 0 | | **0.1** |
| 3116 | 182950 | 7.80 | **8.50** | 1.3 | ~~0~~ | 0 | | **1.3** |
| 3117 | | Interpolated | **13.00** | 1.6 | ~~26~~ | 24 | | **26** |
| 3118 | 209080 | 9.00 | **17.50** | 1.9 | ~~73~~ | 33 | | **35** |
| 3119 | 222000 | 4.95 | **22.45** | 2.1 | ~~134~~ | 40 | 720 | **762** |

Contour Area digitized from gis, top dam and water verified+- by field survey

**DETENTION POND 103**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**
**Improved Pond**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | New WQCV 6" Orf FL 3252 (CFS) | New 12" Orf FL 3256 | New 18" Orf FL 3258 (CFS) | New 18" Orf FL 3263 (CFS) | Raised 60" Dia Riser Weir Rim 3267 (CFS) | Raised 60" Dia Riser Orf Rim 3267 (CFS) | Total Structure Flow (CFS) | 25' Spillway 3268 (CFS) | 42"" Dia RCP Outlet Cap. (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3252 | 0 | 0 | **0** | 0.0 | | | | | | 0.0 | | 0 | **0.0** |
| 3253 | 20000 | 0.23 | **0.23** | 0.5 | | | | | | 0.5 | | 9 | **0.5** |
| 3254 | 38000 | 0.67 | **0.90** | 1.3 | | | | | | 1.3 | | 26 | **1.3** |
| 3255 | 71000 | 1.25 | **2.15** | 1.5 | | | | | | 1.5 | | 50 | **1.5** |
| 3256 | 77000 | 1.70 | **3.85** | 1.8 | 0.0 | | | | | 1.8 | | 74 | **1.8** |
| 3257 | | Interpolated | **5.30** | 2.1 | 2.0 | | | | | 4.1 | | 90 | **4.1** |
| 3258 | 90000 | 3.83 | **7.68** | 2.3 | 6.0 | 0 | | | | 8.3 | | 108 | **8** |
| 3260 | 102877 | 4.43 | **12.11** | 2.6 | 8.0 | 4 | | | | 14.6 | | 132 | **15** |
| 3262 | 118650 | 5.09 | **17.19** | 3.0 | 9.0 | 9 | | | | 21.0 | | 150 | **21** |
| 3263 | | Interpolated | **20.50** | 3.1 | 9.5 | 13 | 0 | | | 25.6 | | 153 | **26** |
| 3264 | 138725 | 5.91 | **23.10** | 3.3 | 10.2 | 15 | 4 | | | 32.5 | | 167 | **33** |
| 3265 | 147500 | 3.29 | **26.39** | 3.4 | 11.0 | 17 | 9 | 0 | 0 | 40.4 | | 175 | **40** |
| 3266 | 156618 | 3.49 | **29.88** | 3.5 | 11.7 | 19 | 13 | 0 | ~~0~~ | 47.2 | | 182 | **47** |
| 3267 | 167000 | 3.71 | **33.59** | 3.7 | 12.1 | 21 | 15 | ~~0~~ | 0 | 51.8 | | 189 | **52** |
| 3268 | 177705 | 3.96 | **37.55** | 3.8 | 12.9 | 22 | 17 | 50 | ~~94~~ | 105.7 | 0 | 197 | **106** |
| 3269 | 199900 | 4.33 | **41.88** | 3.9 | 13.5 | 23 | 19 | ~~145~~ | 134 | 193.4 | 75 | 202 | **268** |
| 3269.5 | | 2.20 | **44.10** | 3.9 | 14 | 23.5 | 20 | | 149 | 210.4 | 114 | 207 | **321** |

Contour Area digitized from original design plans for Pond 103
Top of dam raised to 3269.5
Graded bottom below 3260

**DETENTION POND 104**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**
**Improved w minor grading for low flow channel**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | New WQCV 12" Orf FL 3226 (CFS) | 1 - 24" Orf New Riser FL 3230 (CFS) | New 72" Riser Rim 3238 Weir Flow (CFS) | New 72" Riser Rim 3238 Orf Flow (CFS) | Total Theor Structure Flow (CFS) | Capacity of Improved 30" RCP FL 3226 (CFS) | Modified 20' Spill FL 3239 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3226 | 0 | 0 | **0** | 0 | 0 | | | 0.0 | 0 | | **0** |
| 3227 | 7000 | 0.08 | **0.08** | 1.00 | 0.00 | | | 1.0 | 5 | | **1** |
| 3228 | 12000 | 0.22 | **0.30** | 4.50 | 0.00 | | | 4.5 | 16 | | **5** |
| 3229 | 17000 | 0.33 | **0.63** | 6.00 | 0.00 | | | 6.0 | 31 | | **6** |
| 3230 | 23203 | 0.46 | **1.09** | 7.00 | 0.00 | | | 7.0 | 41 | | **7** |
| 3231 | 26326 | 0.57 | **1.66** | 8.00 | 10.00 | | | 18 | 46 | | **18** |
| 3232 | 29761 | 0.64 | **2.31** | 9.00 | 20.00 | | | 29 | 51 | | **29** |
| 3233 | 33300 | 0.72 | **3.03** | 9.50 | 21.00 | | | 31 | 56 | | **31** |
| 3234 | 36943 | 0.81 | **3.84** | 10.50 | 26.00 | | | 37 | 61 | | **37** |
| 3235 | 40611 | 0.89 | **4.73** | 11.00 | 30.00 | | | 41 | 65 | | **41** |
| 3236 | 44989 | 0.98 | **5.71** | 11.50 | 34.00 | | | 46 | 69 | | **46** |
| 3237 | 49338 | 1.08 | **6.79** | 12.20 | 36.00 | | | 48 | 73 | | **48** |
| 3238 | 53895 | 1.18 | **7.98** | 13.00 | 40.00 | 0 | 0 | 53 | 76 | | **53** |
| 3239 | 58653 | 1.29 | **9.27** | 13.20 | 42.00 | 50 | ~~136~~ | 105 | 79 | 0 | **79** |
| 3240 | 63185 | 1.40 | **10.67** | 14.00 | 45.00 | 160 | ~~192~~ | 219 | 83 | 75 | **158** |
| 3240.2 | 64000 | 0.29 | **10.96** | 14.10 | 47.00 | 185 | ~~202~~ | 246 | 84 | 105 | **189** |

Contour Area digitized from original design plans for Pond 104

**DETENTION POND 105**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | 6" Dia Orf. FL 3108.1 (CFS) | 10' Weir 3109 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|
| 3108.1 | 56600 | 0 | **0** | 0 | | **0.0** |
| 3109 | Interpolated | | **1.8** | 0.7 | | **0.7** |
| 3110 | 108900 | 3.61 | **3.61** | 1.2 | 30 | **31.2** |
| 3111 | Interpolated | | **6.30** | 1.5 | 85 | **86.5** |
| 3112 | 143700 | 5.80 | **9.41** | 1.8 | 156 | **157.8** |

Contour Area digitized from gis

**DETENTION POND 106 - PROPOSED NEW POND**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | 6" Orf FL 3253 (CFS) | 12" Orf FL 3259 (CFS) | 24" Orf FL 3262 (CFS) | 24" Orf FL 3262 (CFS) | 60" Dia Riser Weir Rim 3268 (CFS) | 60" Dia Riser Orf Rim 3268 (CFS) | Total Structure Flow (CFS) | 48 RCP Outlet Cap. FL 3252 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3253 | 0 | 0 | **0** | 0.0 | 0 | | | | | 0 | 0 | **0.00** |
| 3254 | 10000 | 0.11 | **0.11** | 0.7 | 0 | | | | | 0.7 | 25 | **0.70** |
| 3256 | 26000 | 0.83 | **0.94** | 1.5 | 0 | | | | | 1.5 | 70 | **1.50** |
| 3258 | 43000 | 1.58 | **2.53** | 2.0 | 0 | | | | | 2.0 | 115 | **2.00** |
| 3259 | Interpoled | | **3.80** | 2.3 | 0 | | | | | 2.3 | 130 | **2.30** |
| 3260 | 75000 | 2.71 | **5.23** | 2.5 | 2 | | | | | 4.5 | 145 | **4.50** |
| 3262 | 90000 | 3.79 | **9.02** | 2.7 | 6 | 0 | 0 | | | 8.7 | 170 | **8.70** |
| 3264 | 110000 | 4.59 | **13.61** | 3.1 | 8 | 15 | 15 | | | 41 | 190 | **41.10** |
| 3266 | 130000 | 5.51 | **19.12** | 3.4 | 10 | 26 | 26 | | | 65 | 210 | **64.90** |
| 3268 | 150000 | 6.43 | **25.55** | 3.6 | 11 | 34 | 34 | 0 | 0 | 83 | 230 | **82.60** |
| 3269 | Interpolated | | **29.00** | 3.7 | 11 | 37 | 37 | 52 | ~~95~~ | 141 | 240 | **140.70** |
| 3270 | 170000 | 7.35 | **32.90** | 4.0 | 12 | 40 | 40 | ~~145~~ | 135 | 231 | 245 | **231.00** |

Contour Area digitized from GIS contours plu engineering judgement for approximation ofpool grading

**DETENTION POND 107 - PROPOSED NEW POND**
**ELEVATION - STORAGE - DISCHARGE DATA**
**COUNTY HEIGHTS DRAINAGE BASIN DESIGN PLAN AMENDMENT**

| | | | | Outlet at Sta 2655, New Riser on Existing Pipe | | | |
|---|---|---|---|---|---|---|---|
| ELEVATION | CONTOUR AREA (SQ FT) | VOL (AC-FT) | **ACCUM VOL (AC-FT)** | 6" Dia Orf FL 3081.5 (CFS) | 30" Ex. Outlet From New Riser Riser Rim 3084 (CFS) | 30" RCP Sta 2175 FL 3085.2 (CFS) | **FINAL DISCHARGE CURVE (CFS)** |
| 3081.5 | 0 | 0 | **0** | 0.0 | 0.0 | 0 | 0 |
| 3082 | 10000 | 0.06 | **0.06** | 0.5 | ~~4.0~~ | 0 | 0.5 |
| 3084 | 42000 | 1.19 | **1.25** | 1.4 | ~~25.0~~ | 0 | 1.4 |
| 3085.2 | Interpolated | | **2.50** | ~~2.0~~ | 38.0 | 0 | 38.0 |
| 3086 | 54000 | 2.20 | **3.46** | ~~2.4~~ | 44.0 | 3 | 47.0 |
| 3088 | 57000 | 2.55 | **6.00** | ~~2.4~~ | 57.0 | 26 | 83.0 |
| 3090 | 62000 | 2.73 | **8.74** | ~~2.6~~ | 67.0 | 39 | 106.0 |

| Subbasin | Area (MI2) |
|---|---|
| Basin-1F | 0.31 |
| Basin-2F | 0.08 |
| Basin 3F | 0.09 |
| Basin 4F | 0.19 |
| Basin 6F | 0.15 |
| Basin 5F | 0.25 |
| Basin 7F | 0.09 |
| Basin 8F E | 0.04 |
| Basin 8F W | 0.16 |
| Basin 9F | 0.09 |
| Basin 10F | 0.14 |
| Basin 11F | 0.18 |
| Basin14F | 0.17 |
| Basin 15F | 0.06 |
| Basin 12F | 0.08 |
| Basin 13F | 0.17 |
| Basin 16F | 0.19 |
| Basin 18F | 0.13 |
| Basin 17F | 0.09 |

| Subbasin | Initial Storage (%) | Max Storage (IN) |
|---|---|---|
| Basin-1F | 0.0 | 0.35 |
| Basin-2F | 0.0 | 0.35 |
| Basin 3F | 0.0 | 0.35 |
| Basin 4F | 0.0 | 0.35 |
| Basin 6F | 0.0 | 0.35 |
| Basin 5F | 0.0 | 0.35 |
| Basin 7F | 0.0 | 0.35 |
| Basin 8F E | 0.0 | 0.35 |
| Basin 8F W | 0.0 | 0.35 |
| Basin 9F | 0.0 | 0.35 |
| Basin 10F | 0.0 | 0.35 |
| Basin 11F | 0.0 | 0.35 |
| Basin14F | 0.0 | 0.35 |
| Basin 15F | 0.0 | 0.35 |
| Basin 12F | 0.0 | 0.35 |
| Basin 13F | 0.0 | 0.4 |
| Basin 16F | 0.0 | 0.35 |
| Basin 18F | 0.0 | 0.4 |
| Basin 17F | 0.0 | 0.4 |

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| Subbasin | Initial Content | Saturated Content | Suction (IN) | Conductivity (IN/HR) | Impervious (%) |
|---|---|---|---|---|---|
| Basin-1F | 0.116 | 0.434 | 3.5 | 0.512 | 30.3 |
| Basin-2F | 0.116 | 0.434 | 3.5 | 0.512 | 41 |
| Basin 3F | 0.116 | 0.434 | 3.5 | 0.512 | 37.8 |
| Basin 4F | 0.116 | 0.434 | 3.5 | 0.512 | 25.6 |
| Basin 6F | 0.116 | 0.434 | 3.5 | 0.512 | 48.3 |
| Basin 5F | 0.116 | 0.434 | 3.539 | 0.510 | 41.6 |
| Basin 7F | 0.116 | 0.434 | 3.5 | 0.512 | 27.8 |
| Basin 8F E | 0.116 | 0.434 | 3.5 | 0.512 | 33.2 |
| Basin 8F W | 0.116 | 0.434 | 3.5 | 0.512 | 25.9 |
| Basin 9F | 0.116 | 0.434 | 3.5 | 0.512 | 30.3 |
| Basin 10F | 0.116 | 0.364 | 3.5 | 0.512 | 25.7 |
| Basin 11F | 0.138 | 0.346 | 6.219 | 0.264 | 31.6 |
| Basin14F | 0.116 | 0.434 | 3.5 | 0.512 | 28.4 |
| Basin 15F | 0.116 | 0.434 | 3.5 | 0.512 | 34.3 |
| Basin 12F | 0.144 | 0.420 | 6.837 | 0.205 | 61.6 |
| Basin 13F | 0.169 | 0.434 | 10.459 | 0.081 | 19.6 |
| Basin 16F | 0.133 | 0.399 | 5.61 | 0.325 | 40.7 |
| Basin 18F | 0.157 | 0.350 | 8.613 | 0.105 | 26.2 |
| Basin 17F | 0.158 | 0.331 | 8.664 | 0.079 | 28.4 |

| Subbasin | Lag Time (HR) | Peaking Coefficient |
|---|---|---|
| Basin-1F | 0.33 | 0.7 |
| Basin-2F | 0.17 | 0.7 |
| Basin 3F | 0.29 | 0.6 |
| Basin 4F | 0.38 | 0.6 |
| Basin 6F | 0.31 | 0.6 |
| Basin 5F | 0.37 | 0.7 |
| Basin 7F | 0.29 | 0.6 |
| Basin 8F E | 0.33 | 0.6 |
| Basin 8F W | 0.91 | 0.5 |
| Basin 9F | 0.29 | 0.6 |
| Basin 10F | 0.69 | 0.6 |
| Basin 11F | 0.46 | 0.6 |
| Basin14F | 0.66 | 0.6 |
| Basin 15F | 0.37 | 0.6 |
| Basin 12F | 0.21 | 0.6 |
| Basin 13F | 0.63 | 0.6 |
| Basin 16F | 0.44 | 0.6 |
| Basin 18F | 0.61 | 0.5 |
| Basin 17F | 0.86 | 0.5 |

## HEC-HMS 3.5 [K:\Projects\Documents\110136 County Heights Drainage Basin Amendment\HMS Amendment\County_Heights_DBDP\...

| Reach | Time Step Method | Length (FT) | Slope (FT/FT) | Manning's n | Invert (FT) | Shape | Diameter (FT) | Width (FT) | Side Slope (xH:1V) | L.B. Manning's n | R.B. Manning's n | Cross Section Table |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Automatic Fixed Interval | 186 | 0.0097 | 0.013 | | Circle | 3.5 | | | | | |
| 2 | Automatic Fixed Interval | 1100 | 0.006 | 0.035 | | Trapezoid | | 9 | 4 | | | |
| 3 | Automatic Fixed Interval | 188 | 0.0071 | 0.013 | 100 | Circle | 2.5 | | | | | |
| 4 | Automatic Fixed Interval | 290 | 0.012 | 0.050 | | Trapezoid | | 65 | 4 | | | |
| 5 | Automatic Fixed Interval | 92 | 0.0125 | 0.013 | | Circle | 3 | | | | | |
| 6 | Automatic Fixed Interval | 1300 | 0.007 | 0.070 | | Trapezoid | | 25 | 4 | | | |
| 7 | Automatic Fixed Interval | 1100 | 0.0067 | 0.035 | | Trapezoid | | 25 | 4 | | | |
| 8 | Automatic Fixed Interval | 2400 | 0.013 | 0.040 | | Trapezoid | | 20 | 4 | | | |
| 9 | Automatic Fixed Interval | 290 | 0.0029 | 0.013 | | Circle | 3 | | | | | |
| 10 | Automatic Fixed Interval | 700 | 0.022 | 0.05 | | Trapezoid | | 25 | 4 | | | |
| 11 | Automatic Fixed Interval | 1200 | 0.005 | 0.035 | | Trapezoid | | 20 | 4 | | | |
| 12 | Automatic Fixed Interval | 900 | 0.005 | 0.035 | | Trapezoid | | 35 | 4 | | | |
| 13 | Automatic Fixed Interval | 150 | 0.005 | 0.013 | | Rectangle | | 24 | | | | |
| 14 | Automatic Fixed Interval | 1930 | 0.01 | 0.035 | | Trapezoid | | 10 | 3 | | | |
| 15 | Automatic Fixed Interval | 236 | 0.0043 | 0.013 | | Rectangle | | 10 | | | | |
| 16 | Automatic Fixed Interval | 2190 | 0.007 | 0.035 | | Trapezoid | | 10 | 4 | | | |
| 23 | Automatic Fixed Interval | 1500 | 0.006 | 0.013 | | Circle | 4.5 | | | | | |
| 17 | Automatic Fixed Interval | 750 | 0.005 | 0.043 | | Eight Point | | | | 0.035 | 0.035 | Element 17 Composite |
| 17A | Automatic Fixed Interval | 1000 | 0.0065 | 0.045 | | Eight Point | | | | 0.035 | 0.035 | Element 17A Comp... |
| 18 | Automatic Fixed Interval | 1650 | 0.007 | 0.045 | | Eight Point | | | | 0.035 | 0.035 | Element 18 Composite |
| 19 | Automatic Fixed Interval | 1400 | 0.007 | 0.045 | | Eight Point | | | | 0.035 | 0.035 | Element 19 Composite |
| 20 | Automatic Fixed Interval | 168 | 0.002 | 0.013 | | Rectangle | | 28 | | | | |
| 50 | Automatic Fixed Interval | 150 | 0.0066 | 0.013 | | Circle | 4 | | | | | |
| 51 | Automatic Fixed Interval | 200 | 0.015 | 0.013 | | Circle | 2.5 | | | | | |
| 52 | Automatic Fixed Interval | 1900 | 0.004 | 0.035 | | Trapezoid | | 12 | 4 | | | |
| 53 | Automatic Fixed Interval | 1600 | 0.005 | 0.035 | | Trapezoid | | 20 | 6 | | | |
| 21 | Automatic Fixed Interval | 1330 | 0.0083 | 0.045 | | Eight Point | | | | 0.035 | 0.035 | Element 21 Exist |
| 22 | Automatic Fixed Interval | 2200 | 0.007 | 0.042 | | Eight Point | | | | 0.035 | 0.035 | Element 22 Composite |

Project: County Heights DBDP Simulation Run: DBDPA 2 Yr

Start of Run: 01Jun2012, 00:00 Basin Model: DBDP Amendment
End of Run: 02Jun2012, 01:00 Meteorologic Model: 2 Year Storm
Compute Time: 20Sep2012, 14:44:28 Control Specifications: Control 1

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 1.70 | 01Jun2012, 02:35 | 0.14 |
| 10 | 0.40 | 1.98 | 01Jun2012, 06:35 | 0.16 |
| 11 | 0.40 | 1.98 | 01Jun2012, 06:50 | 0.16 |
| 12 | 0.49 | 17.24 | 01Jun2012, 01:00 | 0.17 |
| 13 | 1.16 | 45.33 | 01Jun2012, 01:00 | 0.18 |
| 14 | 1.16 | 44.55 | 01Jun2012, 01:05 | 0.18 |
| 15 | 1.20 | 51.13 | 01Jun2012, 01:05 | 0.19 |
| 16 | 1.36 | 56.61 | 01Jun2012, 01:15 | 0.19 |
| 17 | 1.45 | 65.73 | 01Jun2012, 01:15 | 0.19 |
| 17A | 1.45 | 65.14 | 01Jun2012, 01:20 | 0.19 |
| 18 | 1.59 | 78.22 | 01Jun2012, 01:30 | 0.19 |
| 19 | 1.59 | 77.60 | 01Jun2012, 01:35 | 0.19 |
| 2 | 0.31 | 1.70 | 01Jun2012, 02:45 | 0.14 |
| 20 | 1.77 | 94.20 | 01Jun2012, 01:35 | 0.20 |
| 21 | 2.08 | 101.01 | 01Jun2012, 01:40 | 0.19 |
| 22 | 2.08 | 99.28 | 01Jun2012, 01:45 | 0.19 |
| 23 | 0.09 | 18.52 | 01Jun2012, 00:55 | 0.24 |
| 3 | 0.39 | 6.41 | 01Jun2012, 01:10 | 0.17 |
| 4 | 0.39 | 6.41 | 01Jun2012, 01:15 | 0.17 |
| 5 | 0.48 | 6.47 | 01Jun2012, 02:35 | 0.19 |
| 50 | 0.17 | 0.29 | 01Jun2012, 04:30 | 0.06 |
| 51 | 0.23 | 0.32 | 01Jun2012, 03:35 | 0.05 |
| 52 | 0.23 | 0.32 | 01Jun2012, 04:40 | 0.05 |
| 53 | 0.31 | 7.70 | 01Jun2012, 01:40 | 0.14 |
| 6 | 0.48 | 6.47 | 01Jun2012, 02:50 | 0.19 |
| 7 | 0.48 | 6.47 | 01Jun2012, 03:00 | 0.19 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 2.37 | 01Jun2012, 03:00 | 0.21 |
| 9 | 0.40 | 1.98 | 01Jun2012, 06:25 | 0.17 |
| Basin 10F | 0.14 | 14.31 | 01Jun2012, 01:15 | 0.22 |
| Basin 11F | 0.18 | 33.16 | 01Jun2012, 01:05 | 0.28 |
| Basin 12F | 0.08 | 40.33 | 01Jun2012, 00:45 | 0.47 |
| Basin 13F | 0.17 | 20.46 | 01Jun2012, 01:15 | 0.23 |
| Basin14F | 0.17 | 17.93 | 01Jun2012, 01:15 | 0.22 |
| Basin 15F | 0.06 | 11.55 | 01Jun2012, 01:00 | 0.26 |
| Basin 16F | 0.19 | 38.96 | 01Jun2012, 01:05 | 0.32 |
| Basin 17F | 0.09 | 9.79 | 01Jun2012, 01:25 | 0.32 |
| Basin 18F | 0.13 | 16.43 | 01Jun2012, 01:10 | 0.28 |
| Basin-1F | 0.31 | 71.44 | 01Jun2012, 00:55 | 0.24 |
| Basin-2F | 0.08 | 38.72 | 01Jun2012, 00:45 | 0.31 |
| Basin 3F | 0.09 | 22.35 | 01Jun2012, 00:55 | 0.29 |
| Basin 4F | 0.19 | 28.15 | 01Jun2012, 01:00 | 0.20 |
| Basin 5F | 0.25 | 66.05 | 01Jun2012, 01:00 | 0.31 |
| Basin 6F | 0.15 | 44.63 | 01Jun2012, 00:55 | 0.36 |
| Basin 7F | 0.09 | 17.28 | 01Jun2012, 00:55 | 0.22 |
| Basin 8F E | 0.04 | 8.53 | 01Jun2012, 00:55 | 0.26 |
| Basin 8F W | 0.16 | 9.97 | 01Jun2012, 01:30 | 0.21 |
| Basin 9F | 0.09 | 18.55 | 01Jun2012, 00:55 | 0.24 |
| Det. Pond 100 D | 0.40 | 1.98 | 01Jun2012, 06:25 | 0.17 |
| Det. Pond 103 D | 0.31 | 1.70 | 01Jun2012, 02:30 | 0.14 |
| Det. Pond 106 | 0.25 | 2.37 | 01Jun2012, 02:35 | 0.22 |
| Det Pond 101 D | 0.48 | 6.47 | 01Jun2012, 02:35 | 0.19 |
| Det Pond 102 D | 0.17 | 0.29 | 01Jun2012, 04:30 | 0.06 |
| Det Pond 104 D | 0.39 | 6.41 | 01Jun2012, 01:10 | 0.17 |
| Det Pond 105 D | 0.23 | 0.32 | 01Jun2012, 03:35 | 0.05 |
| Det Pond 107 D | 0.31 | 7.73 | 01Jun2012, 01:25 | 0.15 |
| J201 | 0.39 | 38.73 | 01Jun2012, 00:45 | 0.17 |
| J202 | 0.48 | 28.23 | 01Jun2012, 00:55 | 0.19 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J203 | 0.40 | 44.63 | 01Jun2012, 00:55 | 0.27 |
| J204 | 0.49 | 17.29 | 01Jun2012, 00:55 | 0.17 |
| J205 | 0.67 | 28.25 | 01Jun2012, 01:00 | 0.19 |
| J206 | 1.16 | 45.48 | 01Jun2012, 01:00 | 0.18 |
| J207 | 1.20 | 51.51 | 01Jun2012, 01:05 | 0.19 |
| J208 | 1.36 | 57.18 | 01Jun2012, 01:05 | 0.19 |
| J209 | 1.45 | 67.24 | 01Jun2012, 01:15 | 0.19 |
| J210 | 1.59 | 79.41 | 01Jun2012, 01:20 | 0.19 |
| J211 | 1.77 | 94.26 | 01Jun2012, 01:35 | 0.20 |
| J212 | 2.08 | 101.64 | 01Jun2012, 01:35 | 0.19 |
| J213 | 2.25 | 111.20 | 01Jun2012, 01:45 | 0.20 |
| J250 | 0.23 | 11.58 | 01Jun2012, 01:00 | 0.11 |
| J251 | 0.31 | 40.33 | 01Jun2012, 00:45 | 0.16 |
| J260 | 0.19 | 38.96 | 01Jun2012, 01:05 | 0.32 |
| J261 | 0.09 | 9.79 | 01Jun2012, 01:25 | 0.32 |
| J262 | 0.13 | 16.43 | 01Jun2012, 01:10 | 0.28 |

Project: County Heights DBDP Simulation Run: DBDPA 10 Yr

| | | | |
|---|---|---|---|
| Start of Run: | 01Jun2012, 00:00 | Basin Model: | DBDP Amendment |
| End of Run: | 02Jun2012, 01:00 | Meteorologic Model: | 10 Year Storm |
| Compute Time: | 20Sep2012, 14:24:01 | Control Specifications: | Control 1 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 15.76 | 01Jun2012, 01:50 | 0.67 |
| 10 | 0.40 | 8.26 | 01Jun2012, 04:50 | 0.64 |
| 11 | 0.40 | 8.26 | 01Jun2012, 05:00 | 0.64 |
| 12 | 0.49 | 77.11 | 01Jun2012, 00:55 | 0.67 |
| 13 | 1.16 | 208.45 | 01Jun2012, 00:55 | 0.73 |
| 14 | 1.16 | 206.57 | 01Jun2012, 01:00 | 0.73 |
| 15 | 1.20 | 236.39 | 01Jun2012, 01:00 | 0.74 |
| 16 | 1.36 | 255.74 | 01Jun2012, 01:05 | 0.74 |
| 17 | 1.45 | 304.40 | 01Jun2012, 01:10 | 0.75 |
| 17A | 1.45 | 303.89 | 01Jun2012, 01:10 | 0.75 |
| 18 | 1.59 | 361.71 | 01Jun2012, 01:15 | 0.76 |
| 19 | 1.59 | 359.14 | 01Jun2012, 01:20 | 0.76 |
| 2 | 0.31 | 15.75 | 01Jun2012, 01:55 | 0.67 |
| 20 | 1.77 | 447.76 | 01Jun2012, 01:20 | 0.79 |
| 21 | 2.08 | 484.05 | 01Jun2012, 01:25 | 0.74 |
| 22 | 2.08 | 481.08 | 01Jun2012, 01:30 | 0.73 |
| 23 | 0.09 | 77.54 | 01Jun2012, 00:55 | 0.86 |
| 3 | 0.39 | 30.52 | 01Jun2012, 01:05 | 0.73 |
| 4 | 0.39 | 30.49 | 01Jun2012, 01:10 | 0.73 |
| 5 | 0.48 | 53.84 | 01Jun2012, 01:25 | 0.77 |
| 50 | 0.17 | 1.12 | 01Jun2012, 04:15 | 0.21 |
| 51 | 0.23 | 9.07 | 01Jun2012, 01:45 | 0.25 |
| 52 | 0.23 | 9.03 | 01Jun2012, 02:00 | 0.24 |
| 53 | 0.31 | 43.42 | 01Jun2012, 01:20 | 0.46 |
| 6 | 0.48 | 53.15 | 01Jun2012, 01:35 | 0.76 |
| 7 | 0.48 | 52.86 | 01Jun2012, 01:40 | 0.76 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 24.10 | 01Jun2012, 02:00 | 0.79 |
| 9 | 0.40 | 8.26 | 01Jun2012, 04:45 | 0.64 |
| Basin 10F | 0.14 | 63.31 | 01Jun2012, 01:15 | 0.86 |
| Basin 11F | 0.18 | 132.29 | 01Jun2012, 01:00 | 1.02 |
| Basin 12F | 0.08 | 120.55 | 01Jun2012, 00:45 | 1.27 |
| Basin 13F | 0.17 | 97.83 | 01Jun2012, 01:10 | 1.02 |
| Basin14F | 0.17 | 76.58 | 01Jun2012, 01:15 | 0.84 |
| Basin 15F | 0.06 | 46.23 | 01Jun2012, 00:55 | 0.90 |
| Basin 16F | 0.19 | 143.89 | 01Jun2012, 01:00 | 1.02 |
| Basin 17F | 0.09 | 37.84 | 01Jun2012, 01:25 | 1.18 |
| Basin 18F | 0.13 | 68.81 | 01Jun2012, 01:10 | 1.10 |
| Basin-1F | 0.31 | 297.64 | 01Jun2012, 00:55 | 0.86 |
| Basin-2F | 0.08 | 132.70 | 01Jun2012, 00:45 | 0.97 |
| Basin 3F | 0.09 | 85.63 | 01Jun2012, 00:50 | 0.94 |
| Basin 4F | 0.19 | 131.08 | 01Jun2012, 00:55 | 0.81 |
| Basin 5F | 0.25 | 238.55 | 01Jun2012, 00:55 | 0.98 |
| Basin 6F | 0.15 | 146.21 | 01Jun2012, 00:50 | 1.05 |
| Basin 7F | 0.09 | 78.51 | 01Jun2012, 00:50 | 0.83 |
| Basin 8F E | 0.04 | 34.19 | 01Jun2012, 00:55 | 0.89 |
| Basin 8F W | 0.16 | 43.96 | 01Jun2012, 01:25 | 0.81 |
| Basin 9F | 0.09 | 80.29 | 01Jun2012, 00:50 | 0.86 |
| Det. Pond 100 D | 0.40 | 8.26 | 01Jun2012, 04:45 | 0.64 |
| Det. Pond 103 D | 0.31 | 15.76 | 01Jun2012, 01:50 | 0.67 |
| Det. Pond 106 | 0.25 | 24.12 | 01Jun2012, 01:50 | 0.79 |
| Det Pond 101 D | 0.48 | 53.86 | 01Jun2012, 01:25 | 0.77 |
| Det Pond 102 D | 0.17 | 1.12 | 01Jun2012, 04:15 | 0.21 |
| Det Pond 104 D | 0.39 | 30.53 | 01Jun2012, 01:05 | 0.73 |
| Det Pond 105 D | 0.23 | 9.08 | 01Jun2012, 01:45 | 0.25 |
| Det Pond 107 D | 0.31 | 43.62 | 01Jun2012, 01:10 | 0.46 |
| J201 | 0.39 | 133.01 | 01Jun2012, 00:45 | 0.74 |
| J202 | 0.48 | 110.46 | 01Jun2012, 00:55 | 0.77 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J203 | 0.40 | 146.46 | 01Jun2012, 00:50 | 0.89 |
| J204 | 0.49 | 78.63 | 01Jun2012, 00:50 | 0.67 |
| J205 | 0.67 | 132.44 | 01Jun2012, 00:55 | 0.78 |
| J206 | 1.16 | 209.54 | 01Jun2012, 00:55 | 0.73 |
| J207 | 1.20 | 237.32 | 01Jun2012, 01:00 | 0.74 |
| J208 | 1.36 | 260.87 | 01Jun2012, 01:00 | 0.75 |
| J209 | 1.45 | 312.41 | 01Jun2012, 01:05 | 0.75 |
| J210 | 1.59 | 365.46 | 01Jun2012, 01:10 | 0.76 |
| J211 | 1.77 | 448.58 | 01Jun2012, 01:20 | 0.79 |
| J212 | 2.08 | 491.18 | 01Jun2012, 01:20 | 0.74 |
| J213 | 2.25 | 556.32 | 01Jun2012, 01:30 | 0.76 |
| J250 | 0.23 | 46.33 | 01Jun2012, 00:55 | 0.39 |
| J251 | 0.31 | 120.56 | 01Jun2012, 00:45 | 0.51 |
| J260 | 0.19 | 143.89 | 01Jun2012, 01:00 | 1.02 |
| J261 | 0.09 | 37.84 | 01Jun2012, 01:25 | 1.18 |
| J262 | 0.13 | 68.81 | 01Jun2012, 01:10 | 1.10 |

Project: County Heights DBDP Simulation Run: DBDPA 100 Yr

Start of Run: 01Jun2012, 00:00 Basin Model: DBDP Amendment
End of Run: 02Jun2012, 01:00 Meteorologic Model: 100 Year Storm
Compute Time: 20Sep2012, 13:54:37 Control Specifications: Control 1

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 1 | 0.31 | 41.55 | 01Jun2012, 01:40 | 1.60 |
| 10 | 0.40 | 42.27 | 01Jun2012, 03:40 | 1.56 |
| 11 | 0.40 | 42.26 | 01Jun2012, 03:50 | 1.55 |
| 12 | 0.49 | 160.65 | 01Jun2012, 00:55 | 1.60 |
| 13 | 1.16 | 443.92 | 01Jun2012, 00:55 | 1.66 |
| 14 | 1.16 | 438.99 | 01Jun2012, 01:00 | 1.66 |
| 15 | 1.20 | 501.55 | 01Jun2012, 01:00 | 1.67 |
| 16 | 1.36 | 551.59 | 01Jun2012, 01:05 | 1.68 |
| 17 | 1.45 | 667.82 | 01Jun2012, 01:05 | 1.69 |
| 17A | 1.45 | 658.00 | 01Jun2012, 01:10 | 1.69 |
| 18 | 1.59 | 782.40 | 01Jun2012, 01:15 | 1.70 |
| 19 | 1.59 | 779.22 | 01Jun2012, 01:15 | 1.70 |
| 2 | 0.31 | 41.52 | 01Jun2012, 01:45 | 1.60 |
| 20 | 1.77 | 990.07 | 01Jun2012, 01:15 | 1.74 |
| 21 | 2.08 | 1056.19 | 01Jun2012, 01:20 | 1.68 |
| 22 | 2.08 | 1047.76 | 01Jun2012, 01:25 | 1.68 |
| 23 | 0.09 | 159.77 | 01Jun2012, 00:55 | 1.84 |
| 3 | 0.39 | 47.45 | 01Jun2012, 01:15 | 1.67 |
| 4 | 0.39 | 47.43 | 01Jun2012, 01:20 | 1.67 |
| 5 | 0.48 | 101.36 | 01Jun2012, 01:20 | 1.69 |
| 50 | 0.17 | 26.28 | 01Jun2012, 02:40 | 1.04 |
| 51 | 0.23 | 35.61 | 01Jun2012, 01:30 | 1.11 |
| 52 | 0.23 | 35.48 | 01Jun2012, 01:40 | 1.11 |
| 53 | 0.31 | 79.82 | 01Jun2012, 01:20 | 1.36 |
| 6 | 0.48 | 101.22 | 01Jun2012, 01:30 | 1.68 |
| 7 | 0.48 | 101.05 | 01Jun2012, 01:35 | 1.68 |

Page 1

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| 8 | 0.25 | 70.73 | 01Jun2012, 01:45 | 1.77 |
| 9 | 0.40 | 42.28 | 01Jun2012, 03:40 | 1.56 |
| Basin 10F | 0.14 | 135.73 | 01Jun2012, 01:15 | 1.85 |
| Basin 11F | 0.18 | 260.76 | 01Jun2012, 01:00 | 2.07 |
| Basin 12F | 0.08 | 211.99 | 01Jun2012, 00:45 | 2.34 |
| Basin 13F | 0.17 | 196.43 | 01Jun2012, 01:10 | 2.09 |
| Basin14F | 0.17 | 165.68 | 01Jun2012, 01:15 | 1.81 |
| Basin 15F | 0.06 | 94.74 | 01Jun2012, 00:55 | 1.89 |
| Basin 16F | 0.19 | 284.05 | 01Jun2012, 01:00 | 2.06 |
| Basin 17F | 0.09 | 72.18 | 01Jun2012, 01:25 | 2.26 |
| Basin 18F | 0.13 | 134.02 | 01Jun2012, 01:10 | 2.17 |
| Basin-1F | 0.31 | 611.93 | 01Jun2012, 00:55 | 1.84 |
| Basin-2F | 0.08 | 246.45 | 01Jun2012, 00:45 | 1.97 |
| Basin 3F | 0.09 | 170.44 | 01Jun2012, 00:50 | 1.93 |
| Basin 4F | 0.19 | 280.20 | 01Jun2012, 00:55 | 1.78 |
| Basin 5F | 0.25 | 472.23 | 01Jun2012, 00:55 | 1.98 |
| Basin 6F | 0.15 | 281.53 | 01Jun2012, 00:50 | 2.06 |
| Basin 7F | 0.09 | 163.07 | 01Jun2012, 00:50 | 1.81 |
| Basin 8F E | 0.04 | 69.91 | 01Jun2012, 00:55 | 1.87 |
| Basin 8F W | 0.16 | 97.26 | 01Jun2012, 01:25 | 1.78 |
| Basin 9F | 0.09 | 164.92 | 01Jun2012, 00:50 | 1.84 |
| Det. Pond 100 D | 0.40 | 42.28 | 01Jun2012, 03:40 | 1.56 |
| Det. Pond 103 D | 0.31 | 41.56 | 01Jun2012, 01:40 | 1.60 |
| Det. Pond 106 | 0.25 | 70.75 | 01Jun2012, 01:40 | 1.77 |
| Det Pond 101 D | 0.48 | 101.38 | 01Jun2012, 01:20 | 1.69 |
| Det Pond 102 D | 0.17 | 26.28 | 01Jun2012, 02:40 | 1.04 |
| Det Pond 104 D | 0.39 | 47.45 | 01Jun2012, 01:15 | 1.67 |
| Det Pond 105 D | 0.23 | 35.62 | 01Jun2012, 01:30 | 1.11 |
| Det Pond 107 D | 0.31 | 80.05 | 01Jun2012, 01:10 | 1.37 |
| J201 | 0.39 | 247.47 | 01Jun2012, 00:45 | 1.67 |
| J202 | 0.48 | 209.21 | 01Jun2012, 00:50 | 1.72 |

| Hydrologic Element | Drainage Area (MI2) | Peak Discharge (CFS) | Time of Peak | Volume (IN) |
|---|---|---|---|---|
| J203 | 0.40 | 282.30 | 01Jun2012, 00:50 | 1.88 |
| J204 | 0.49 | 163.65 | 01Jun2012, 00:50 | 1.60 |
| J205 | 0.67 | 288.34 | 01Jun2012, 01:10 | 1.71 |
| J206 | 1.16 | 445.33 | 01Jun2012, 00:55 | 1.66 |
| J207 | 1.20 | 502.80 | 01Jun2012, 01:00 | 1.67 |
| J208 | 1.36 | 557.15 | 01Jun2012, 01:00 | 1.68 |
| J209 | 1.45 | 672.37 | 01Jun2012, 01:05 | 1.69 |
| J210 | 1.59 | 790.39 | 01Jun2012, 01:10 | 1.70 |
| J211 | 1.77 | 990.91 | 01Jun2012, 01:15 | 1.74 |
| J212 | 2.08 | 1069.51 | 01Jun2012, 01:15 | 1.68 |
| J213 | 2.25 | 1230.83 | 01Jun2012, 01:20 | 1.71 |
| J250 | 0.23 | 94.99 | 01Jun2012, 00:55 | 1.26 |
| J251 | 0.31 | 212.04 | 01Jun2012, 00:45 | 1.43 |
| J260 | 0.19 | 284.05 | 01Jun2012, 01:00 | 2.06 |
| J261 | 0.09 | 72.18 | 01Jun2012, 01:25 | 2.26 |
| J262 | 0.13 | 134.02 | 01Jun2012, 01:10 | 2.17 |

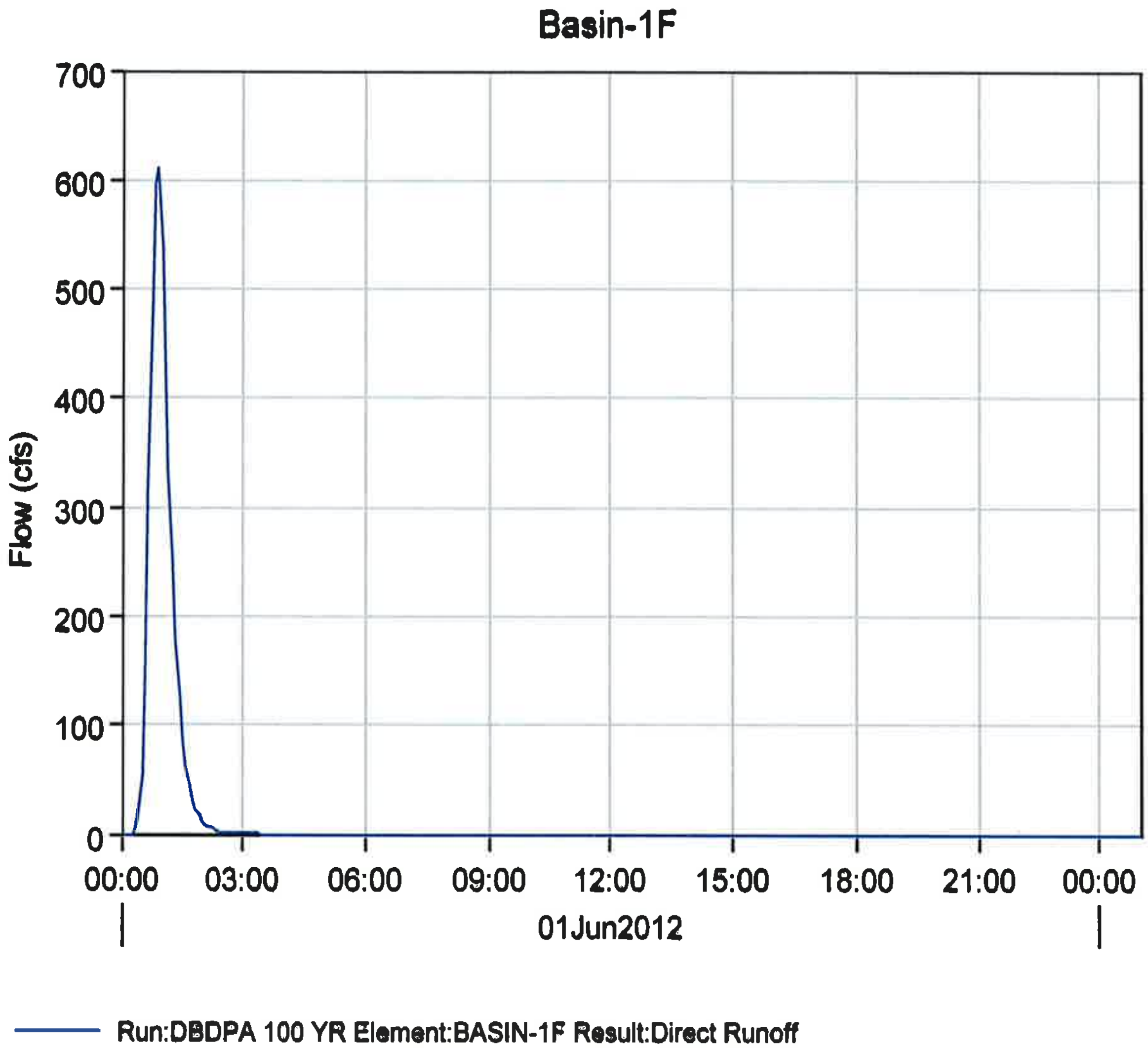
# <u>APPENDIX C</u>

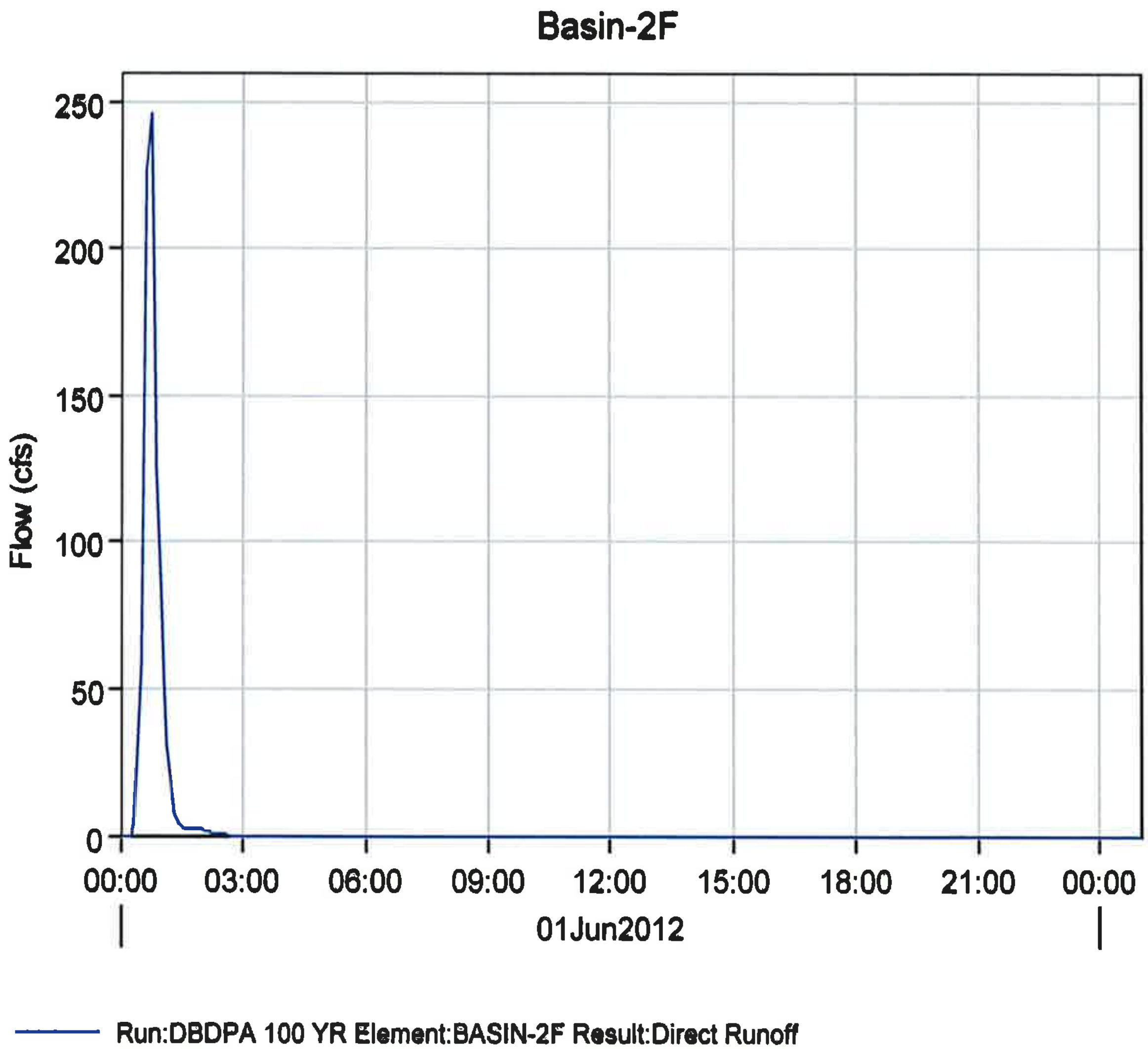
HYDROGRAPHS

FOR

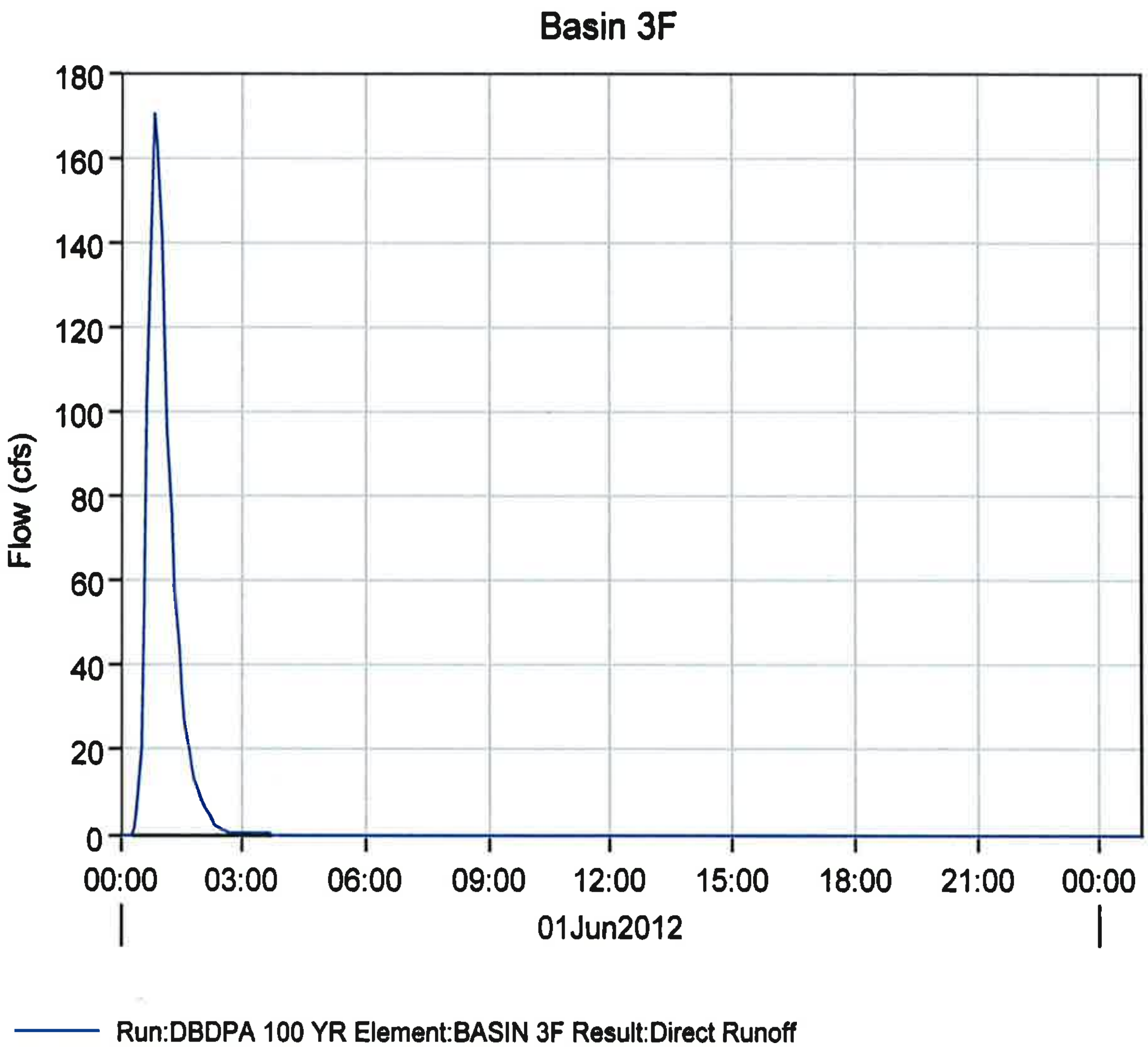
**FUTURE LAND USE**
**AND**
**FUTURE (DBDPA) HYDRAULIC CONDITIONS**

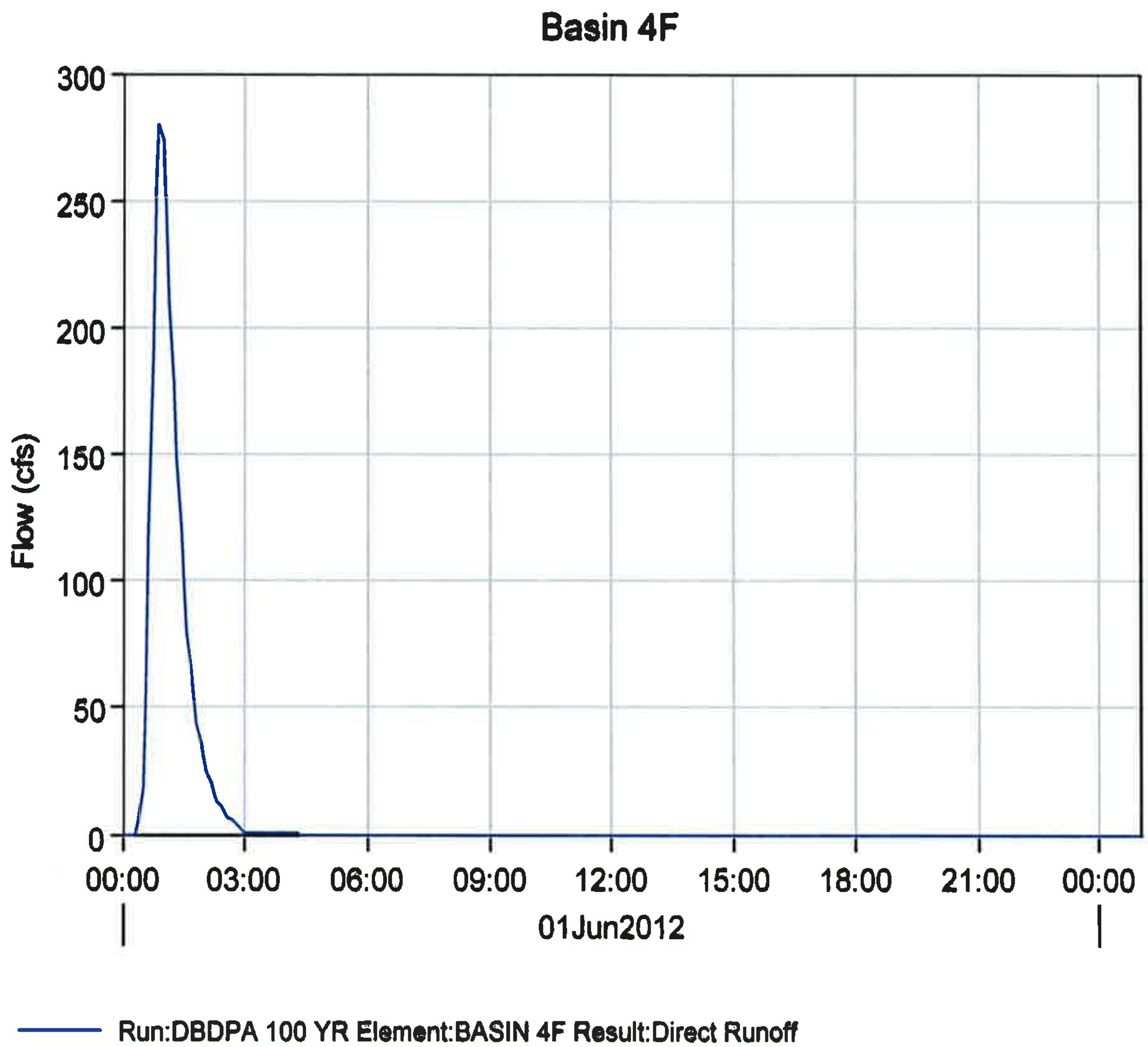
100 Year Hydrographs are included for Sub-basins, Detention Ponds, and Junction Elements.


Basin-1F
Flow (cfs)
700
600
500
400
300
200
100
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN-1F Result:Direct Runoff

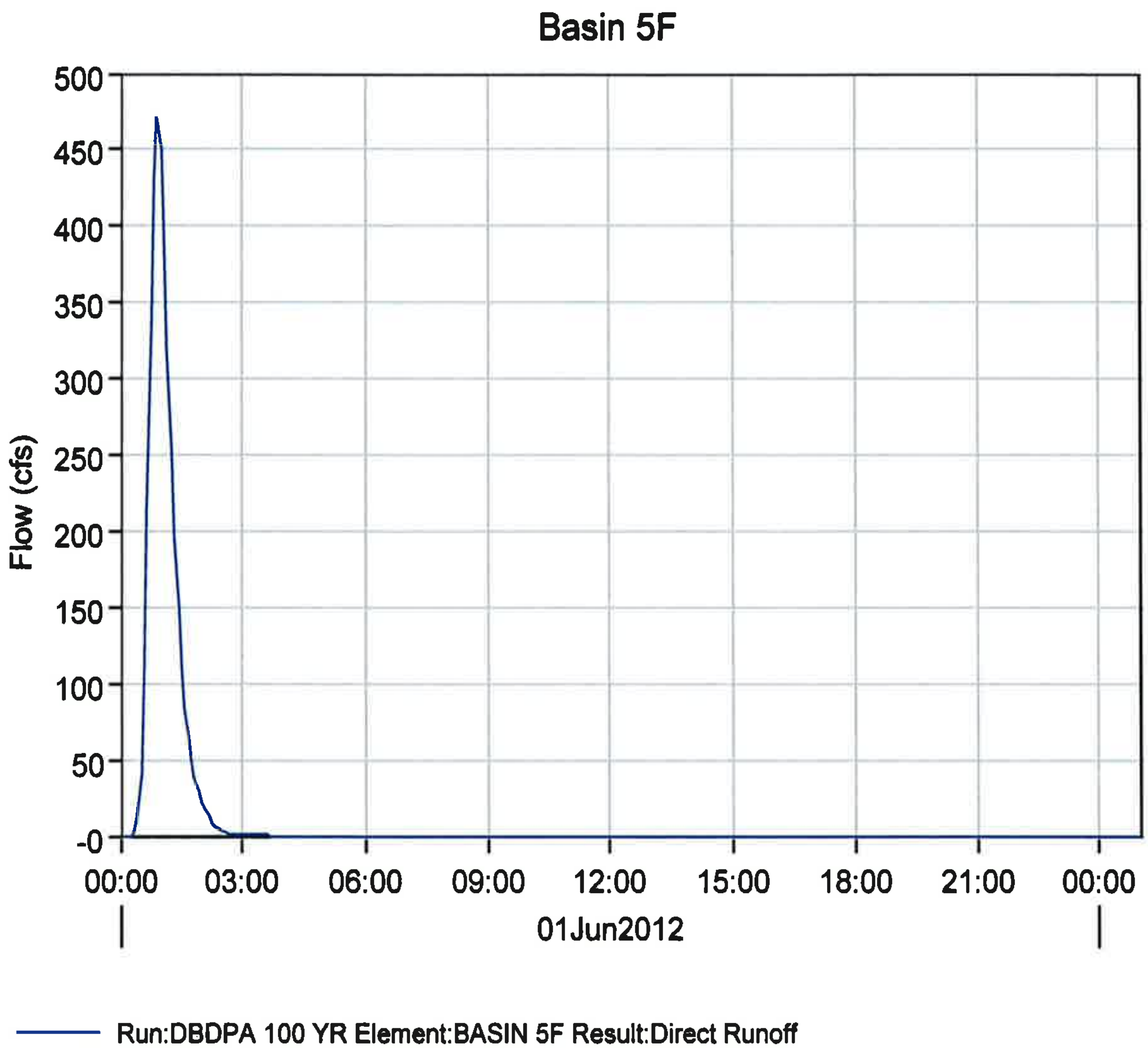


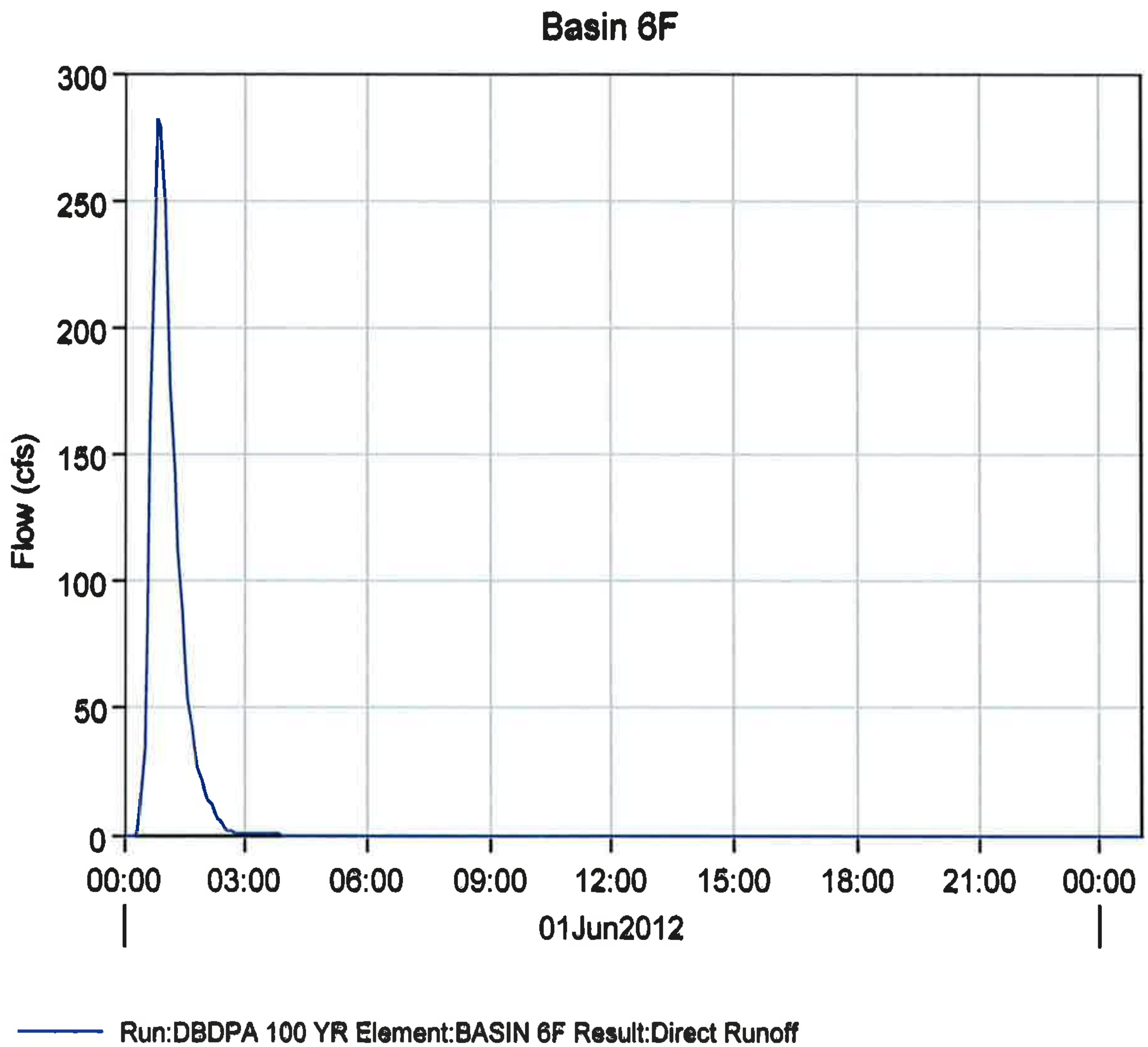
Basin-2F
Flow (cfs)
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN-2F Result:Direct Runoff

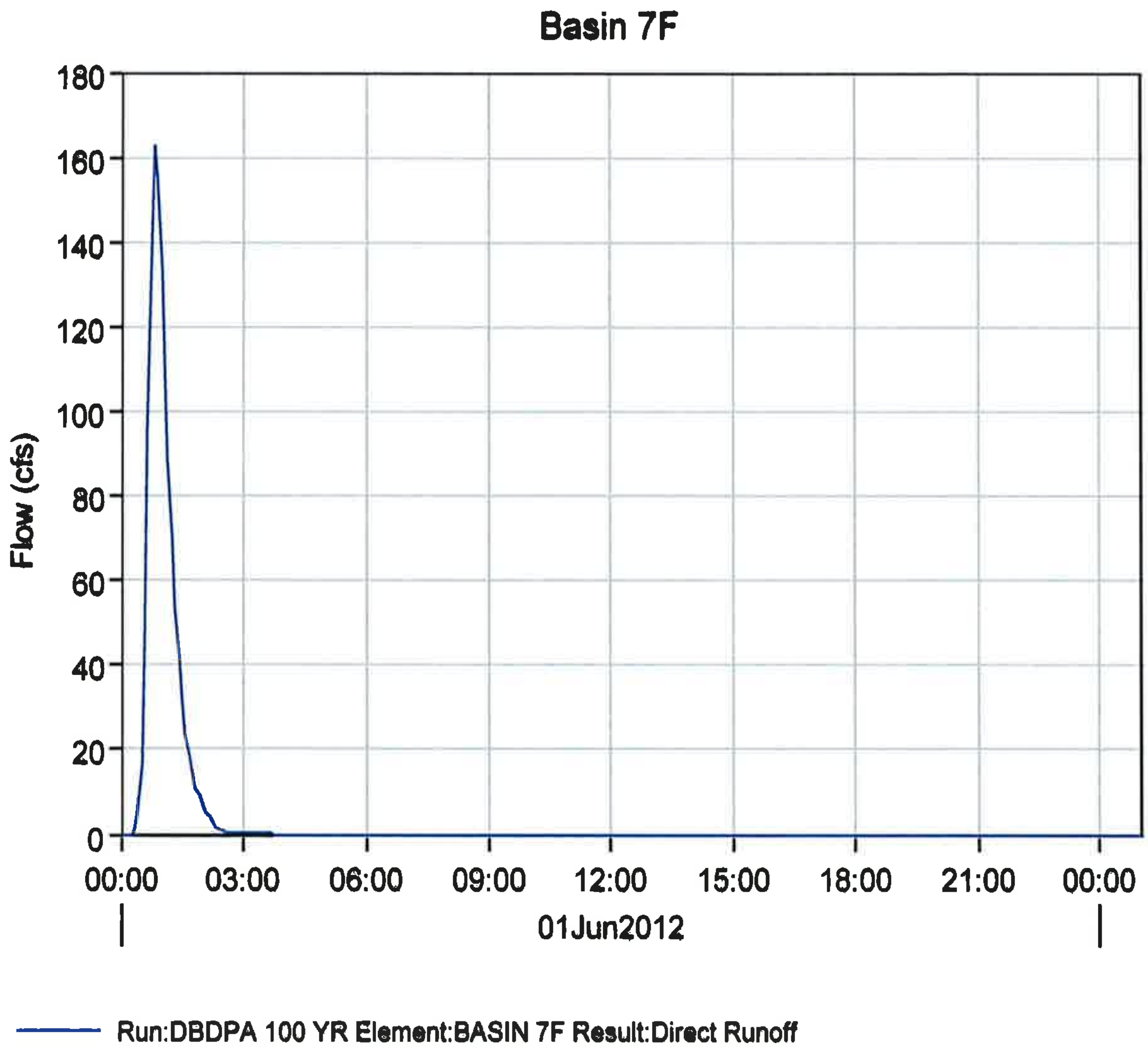

Basin 3F
Flow (cfs)
180
160
140
120
100
80
60
40
20
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 3F Result:Direct Runoff



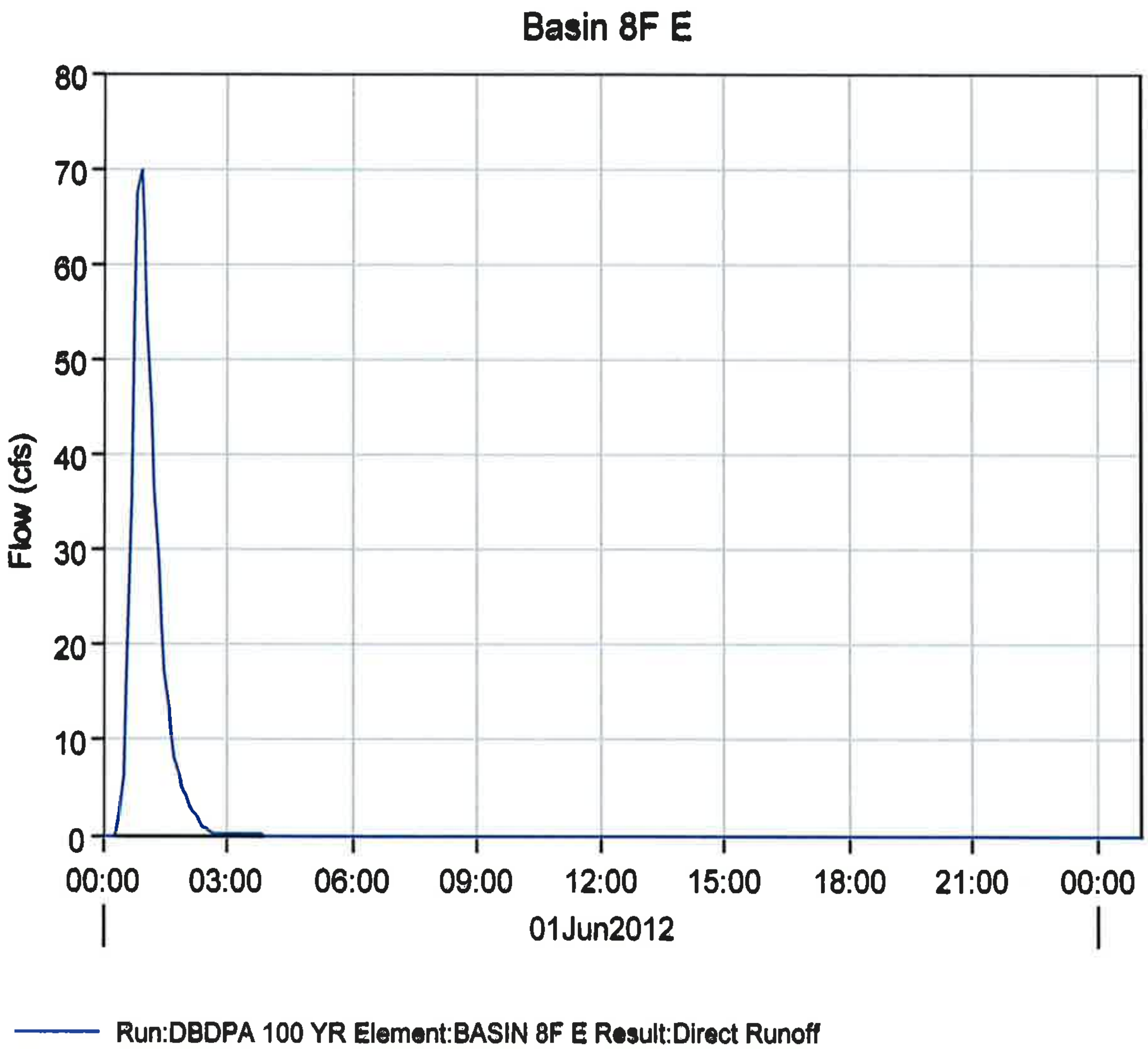
Basin 4F
300
250
200
150
100
50
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 4F Result:Direct Runoff


Basin 5F
Flow (cfs)
500
450
400
350
300
250
200
150
100
50
-0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 5F Result:Direct Runoff

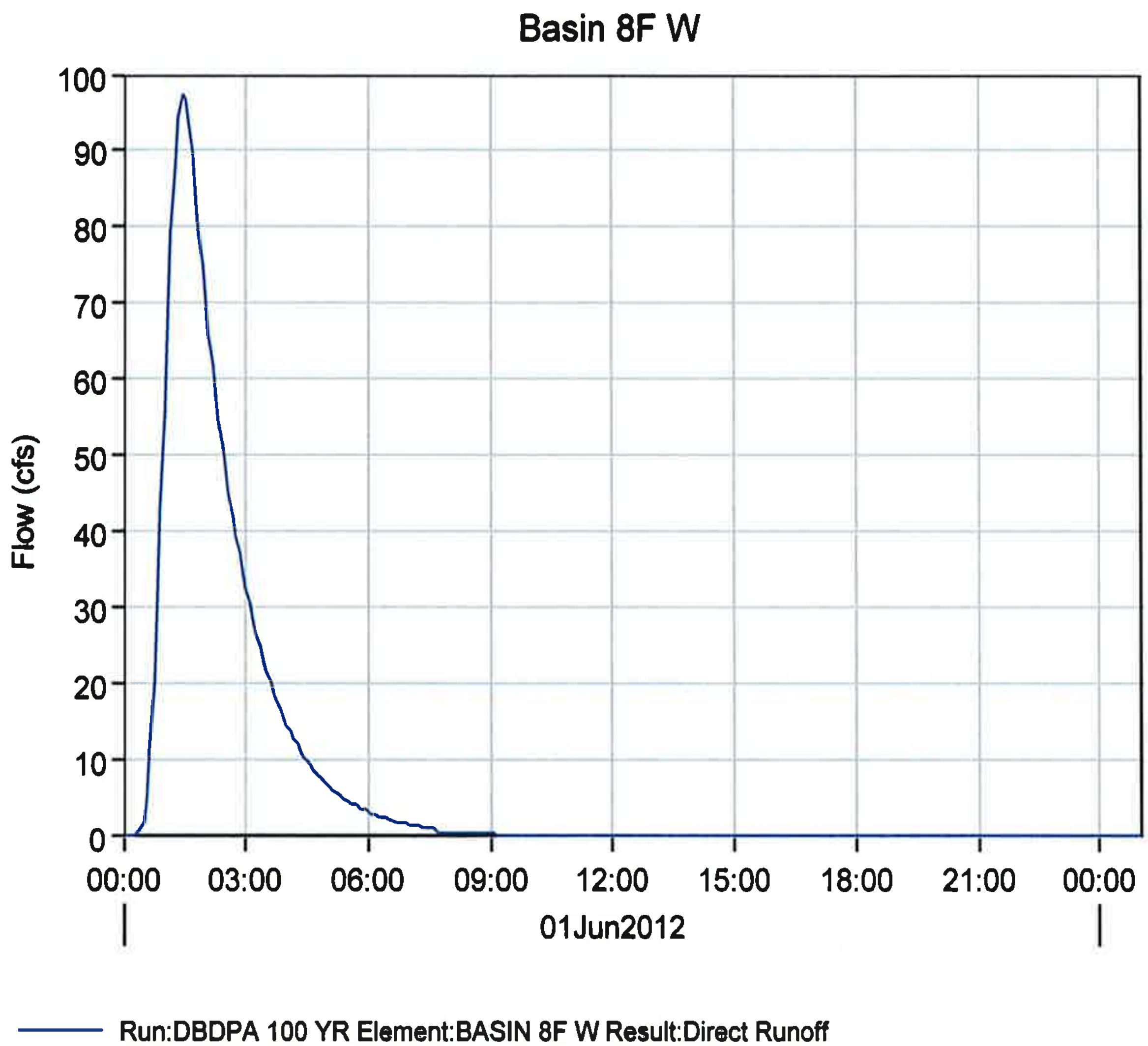

Basin 6F
Flow (cfs)
300
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 6F Result:Direct Runoff

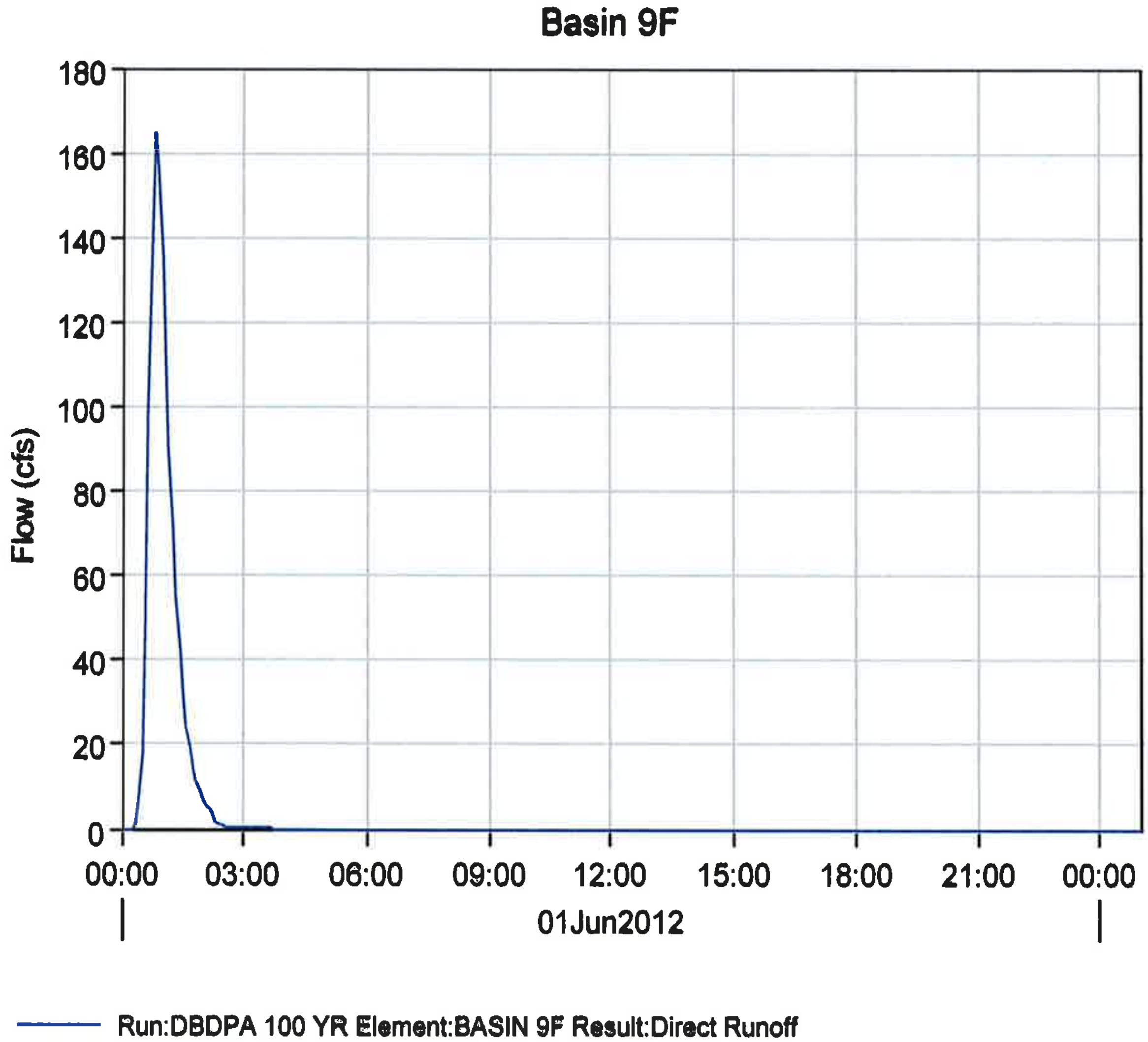


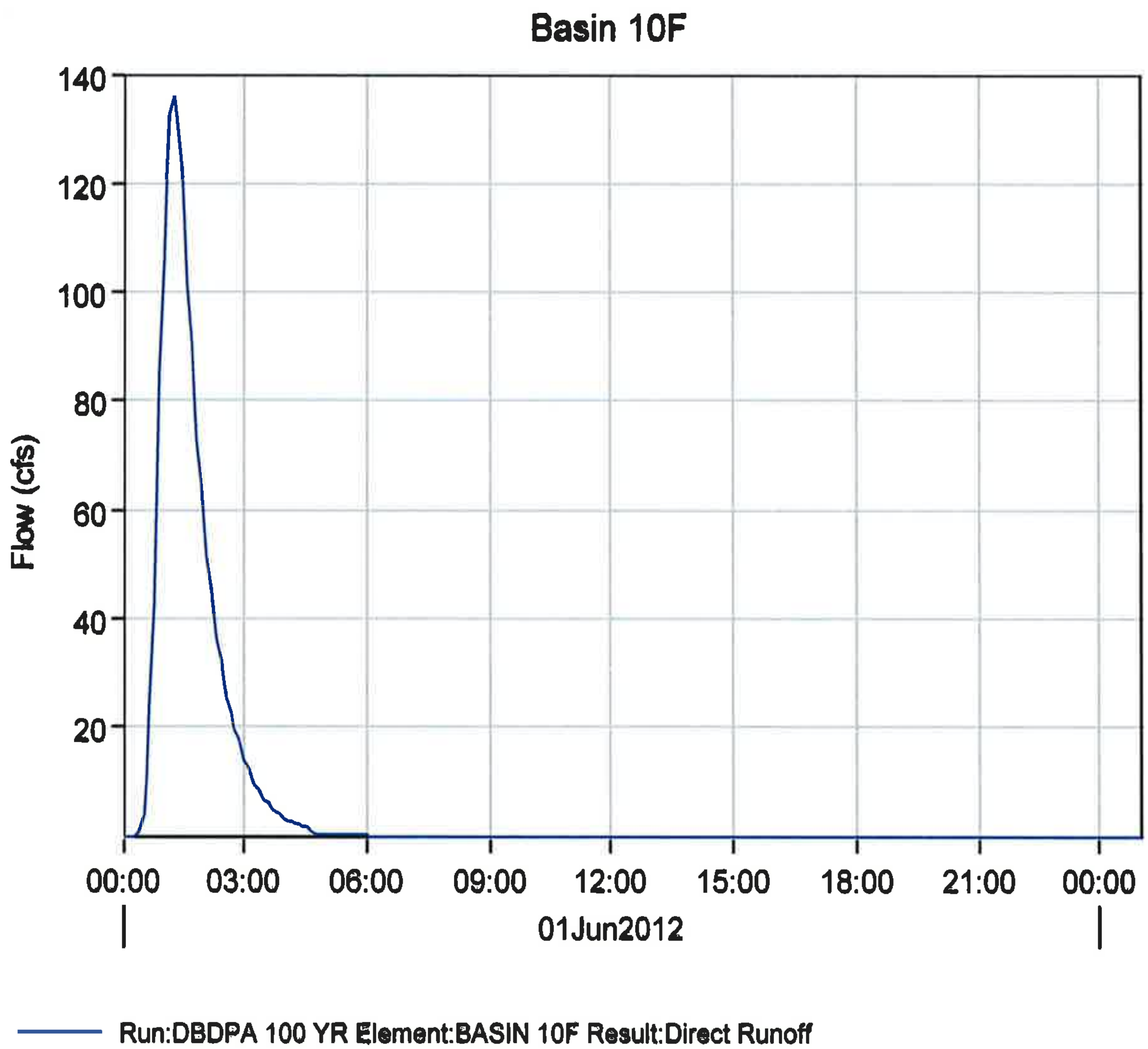
Basin 7F
180
160
140
120
100
80
60
40
20
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 7F Result:Direct Runoff

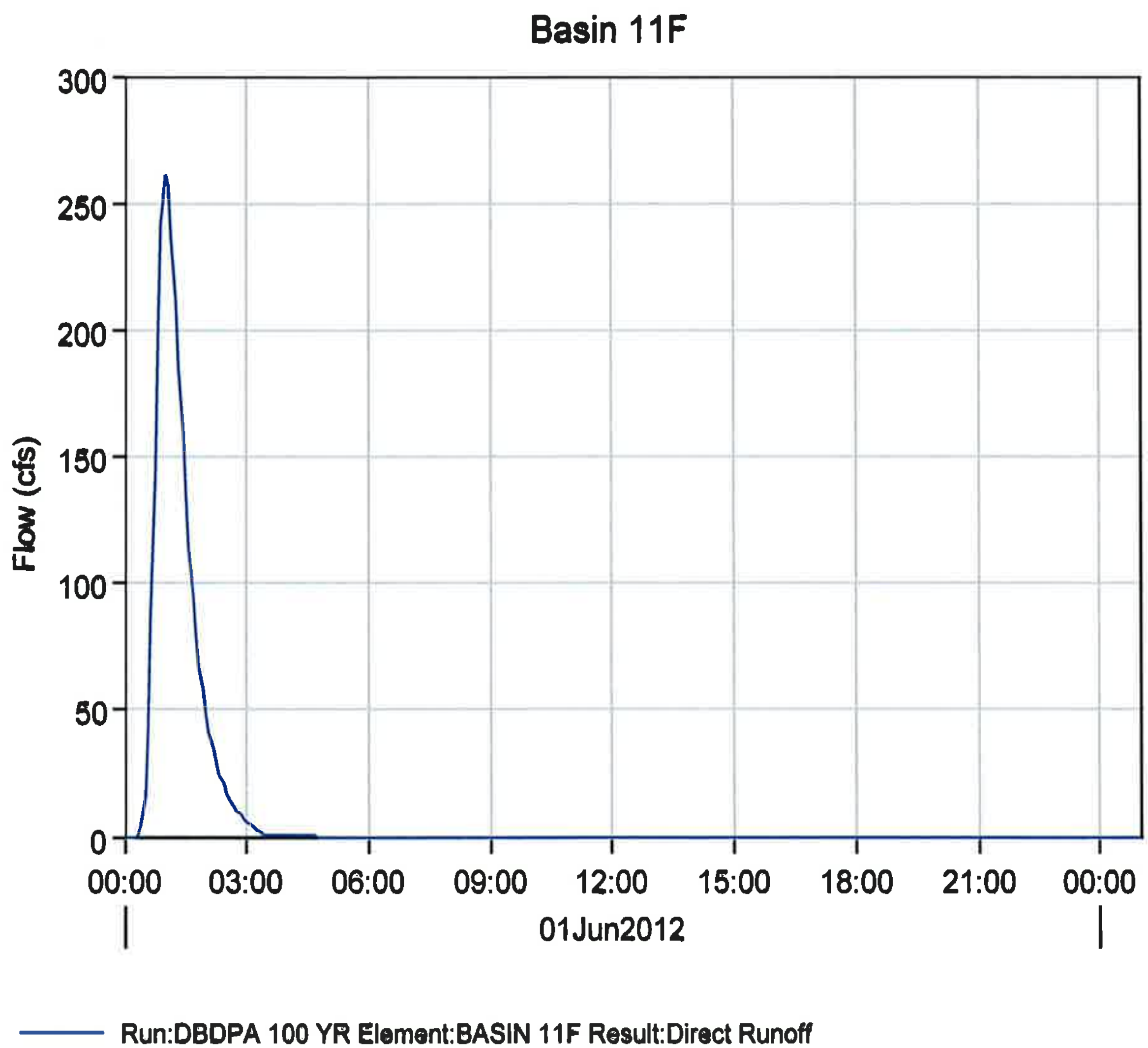


Basin 8F E
Flow (cfs)
80
70
60
50
40
30
20
10
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 8F E Result:Direct Runoff

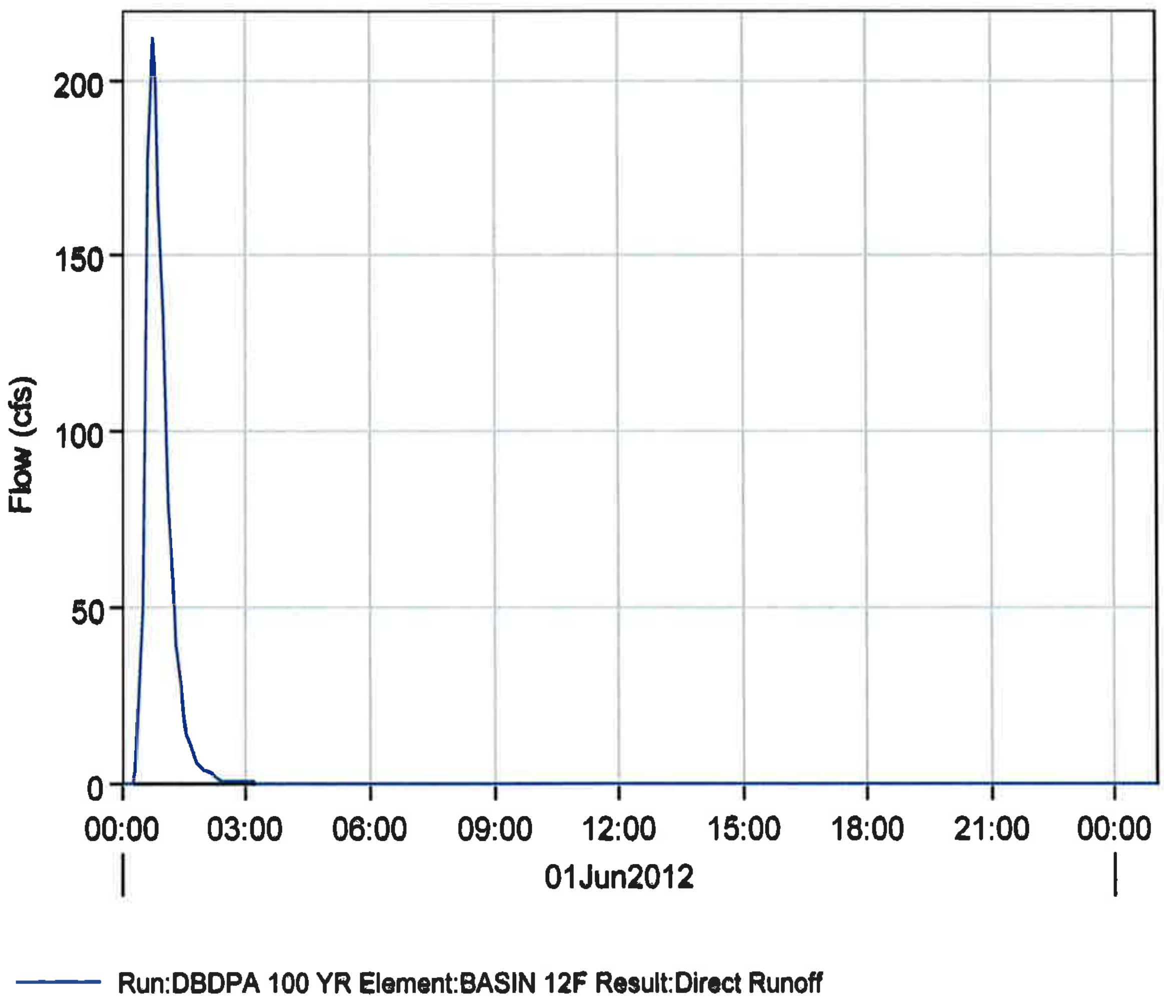

Basin 8F W
Flow (cfs)
100
90
80
70
60
50
40
30
20
10
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 8F W Result:Direct Runoff


Basin 9F
180
160
140
120
100
80
60
40
20
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 9F Result:Direct Runoff

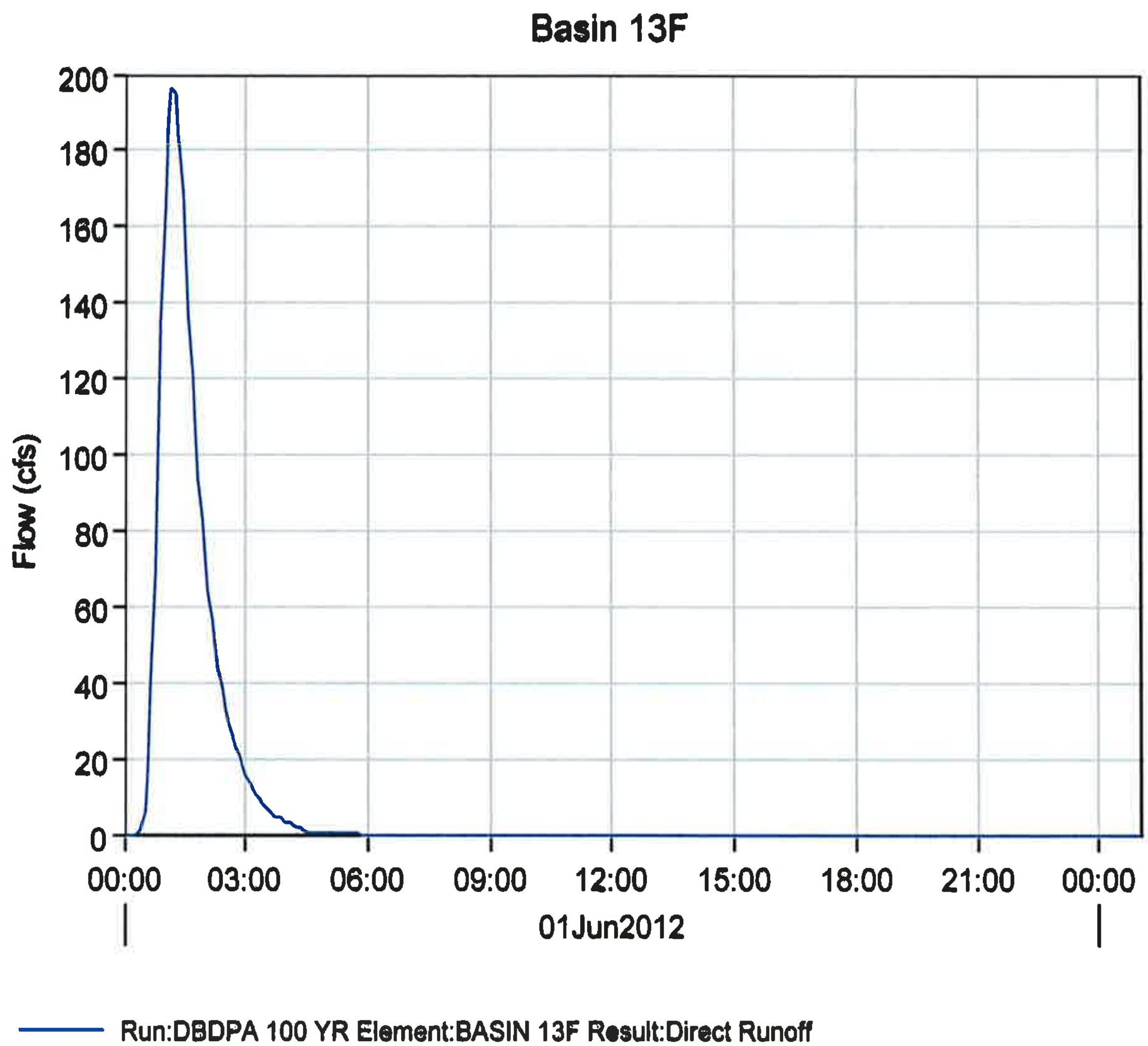

Basin 10F
Flow (cfs)
140
120
100
80
60
40
20
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 10F Result:Direct Runoff

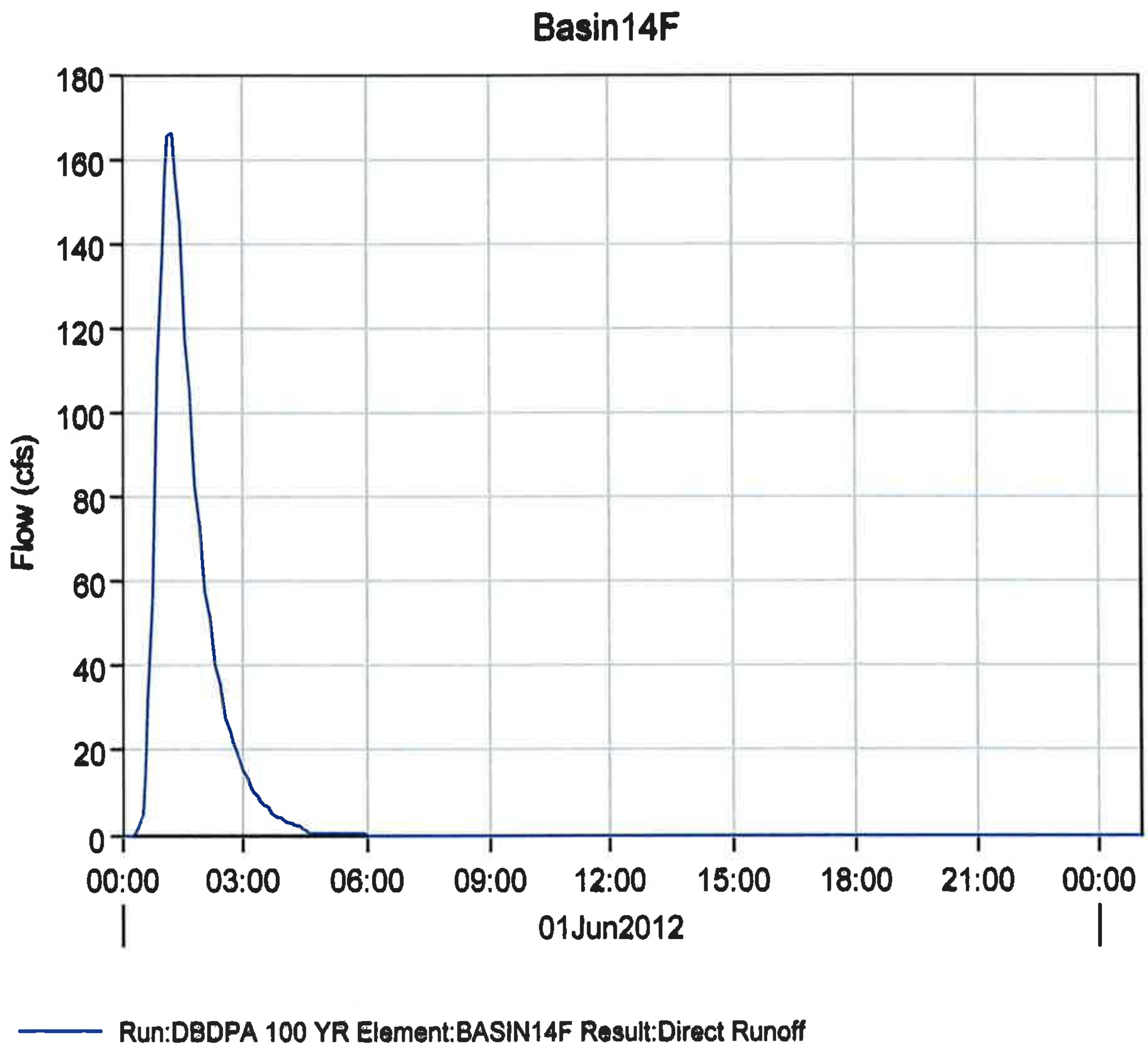

Basin 11F
Flow (cfs)
300
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 11F Result:Direct Runoff

# Basin 12F

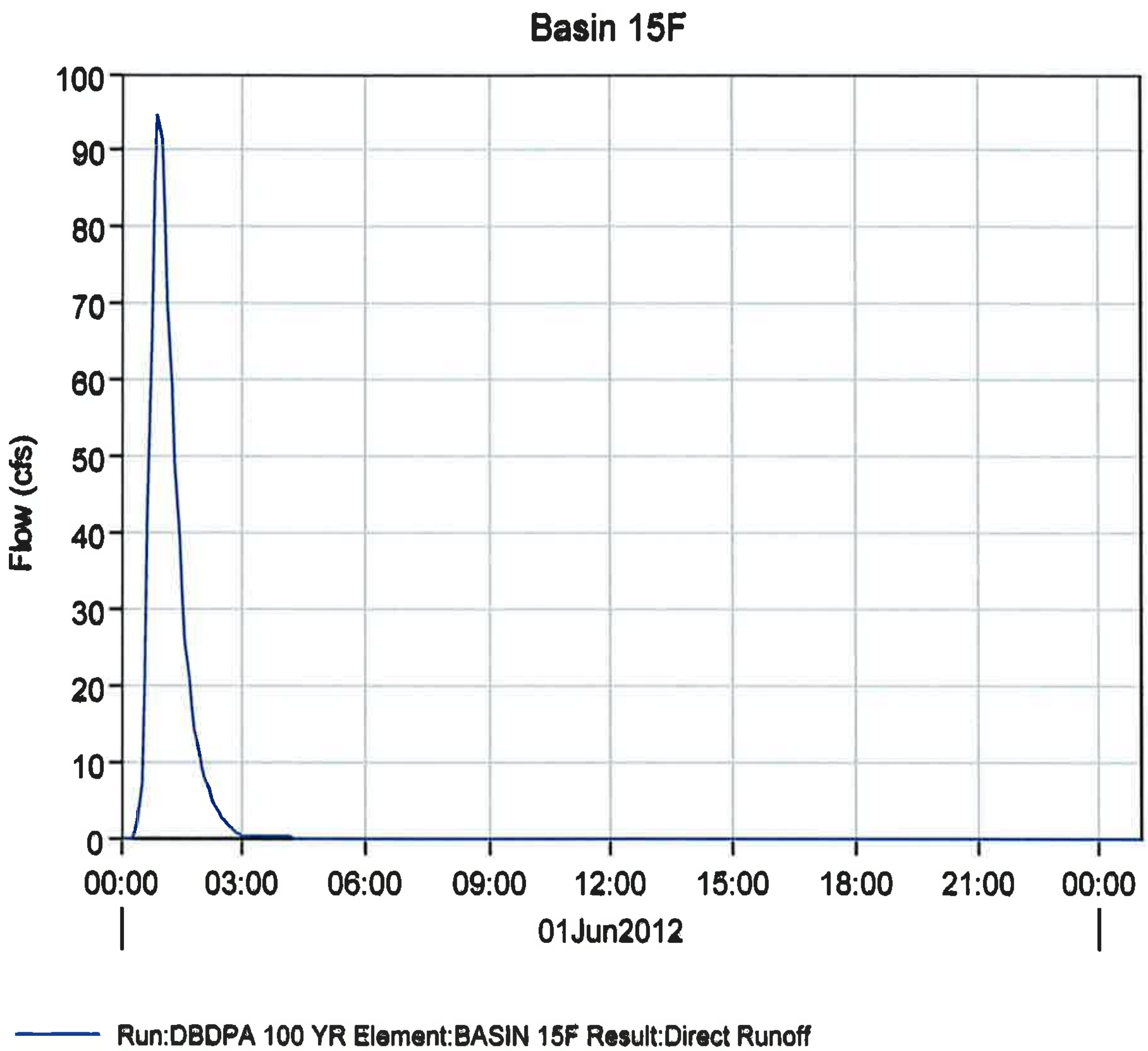


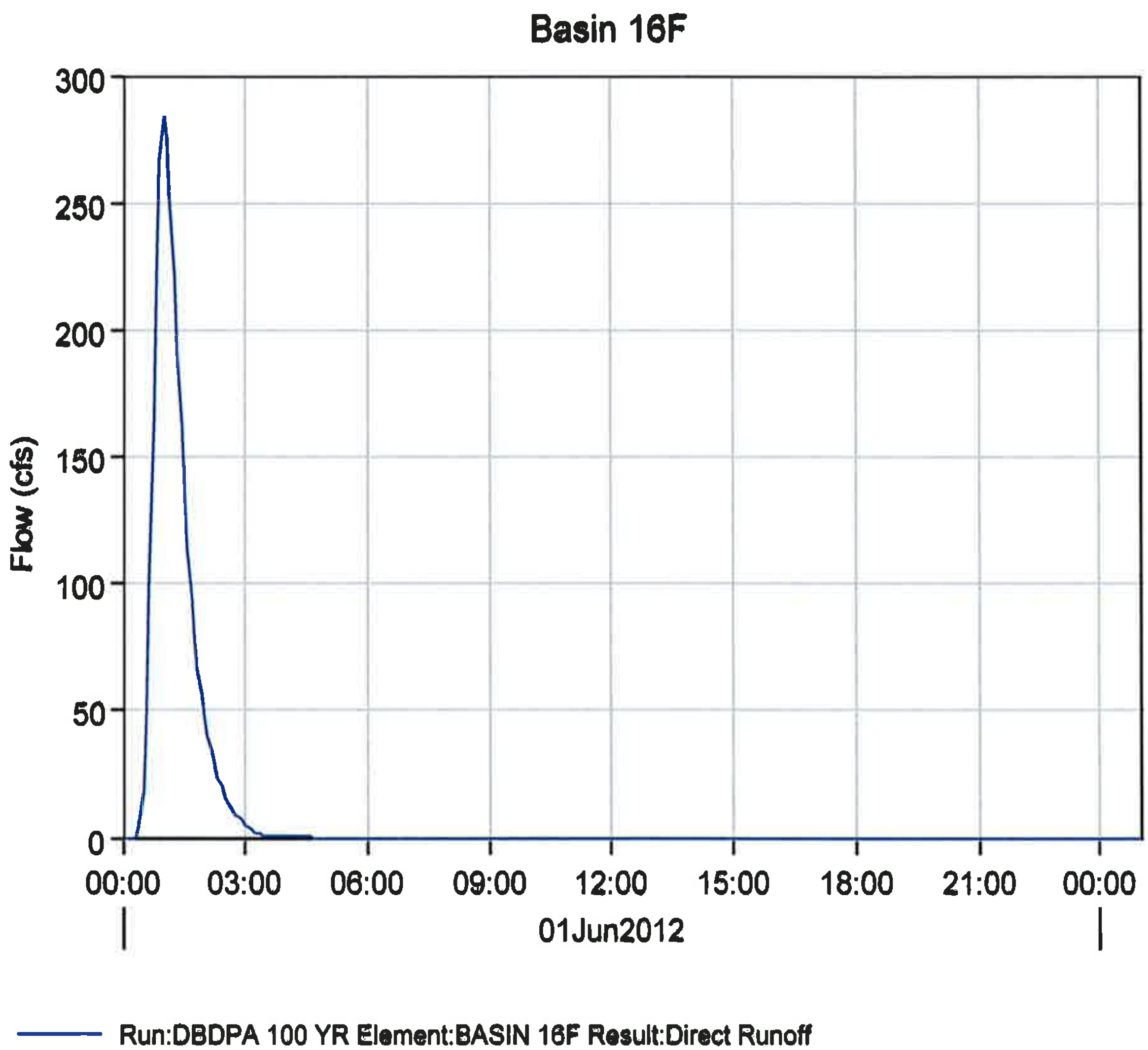
Flow (cfs)
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 12F Result:Direct Runoff


Basin 13F
Flow (cfs)
200
180
160
140
120
100
80
60
40
20
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 13F Result:Direct Runoff

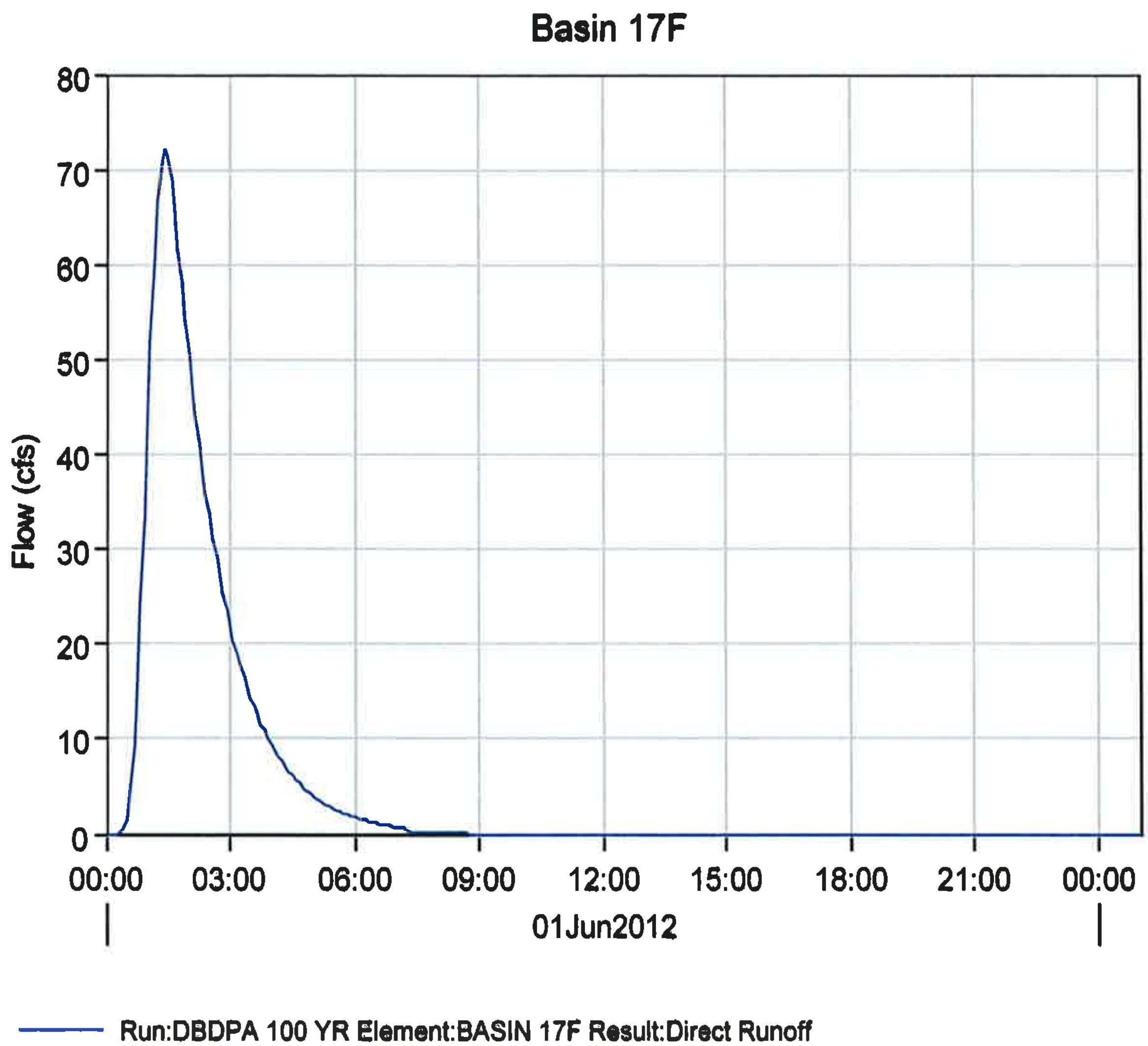


Basin14F
180
160
140
120
100
80
60
40
20
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN14F Result:Direct Runoff

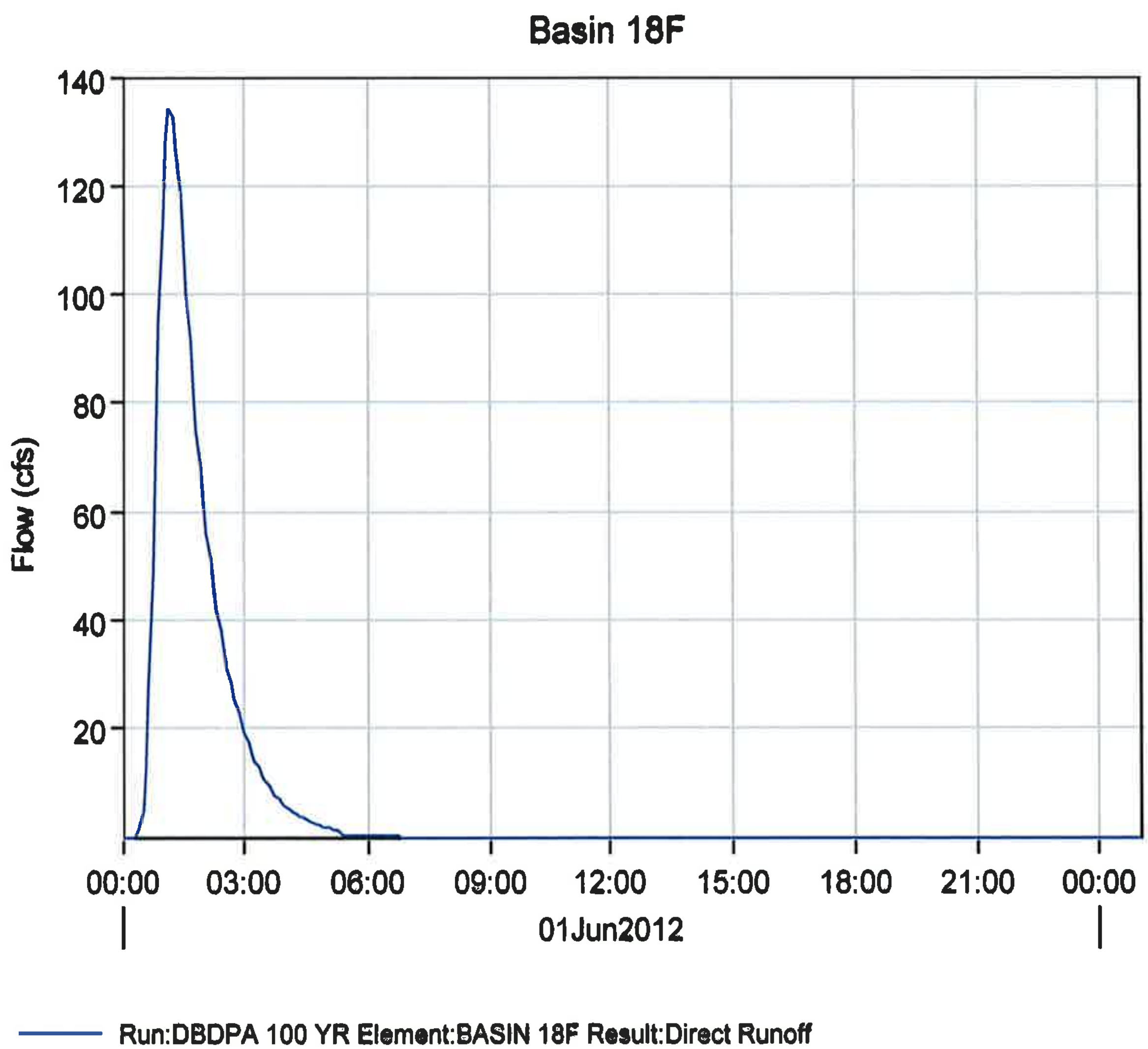

Basin 15F
Flow (cfs)
100
90
80
70
60
50
40
30
20
10
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 15F Result:Direct Runoff



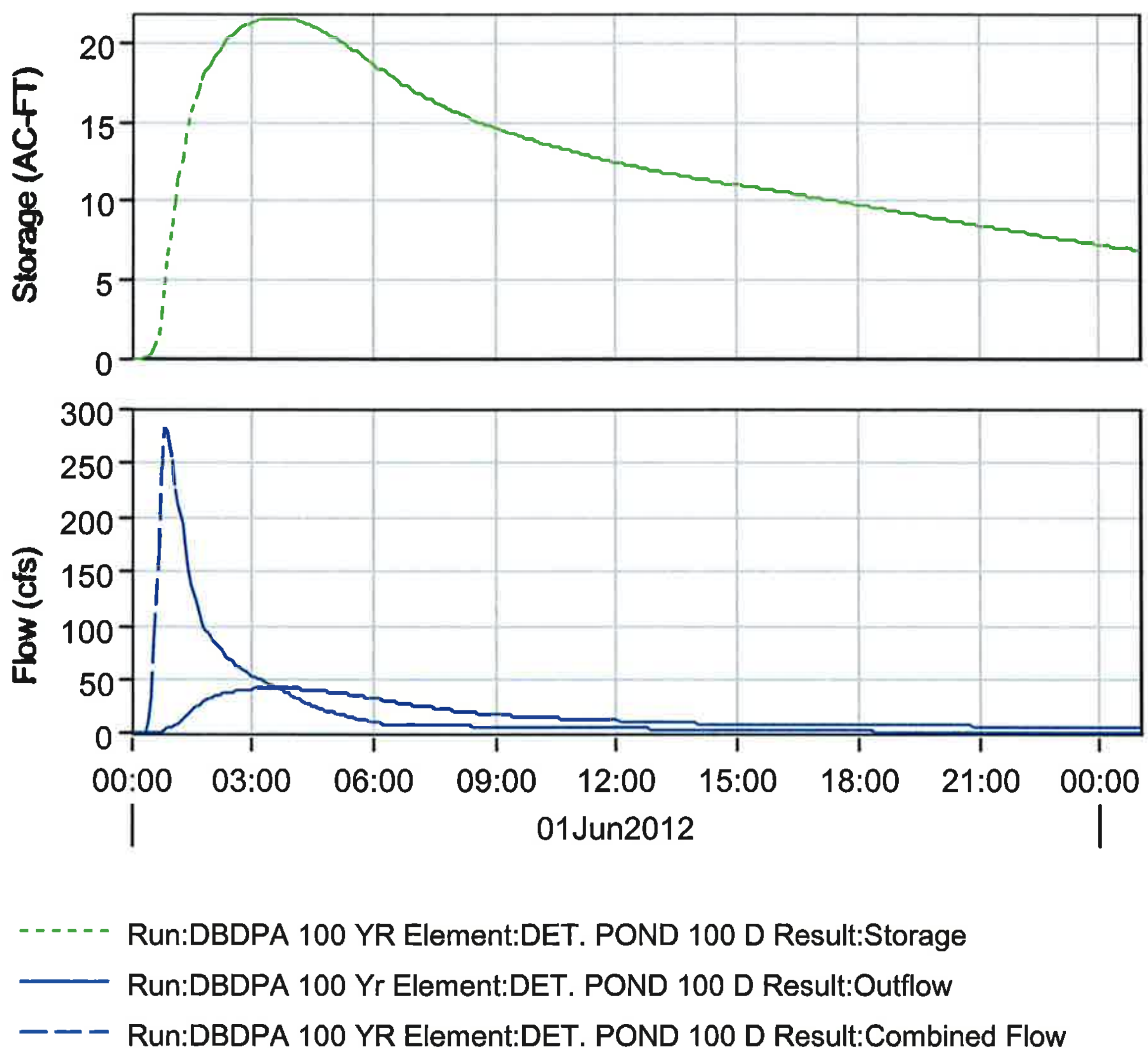
Basin 16F
Flow (cfs)
300
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 16F Result:Direct Runoff

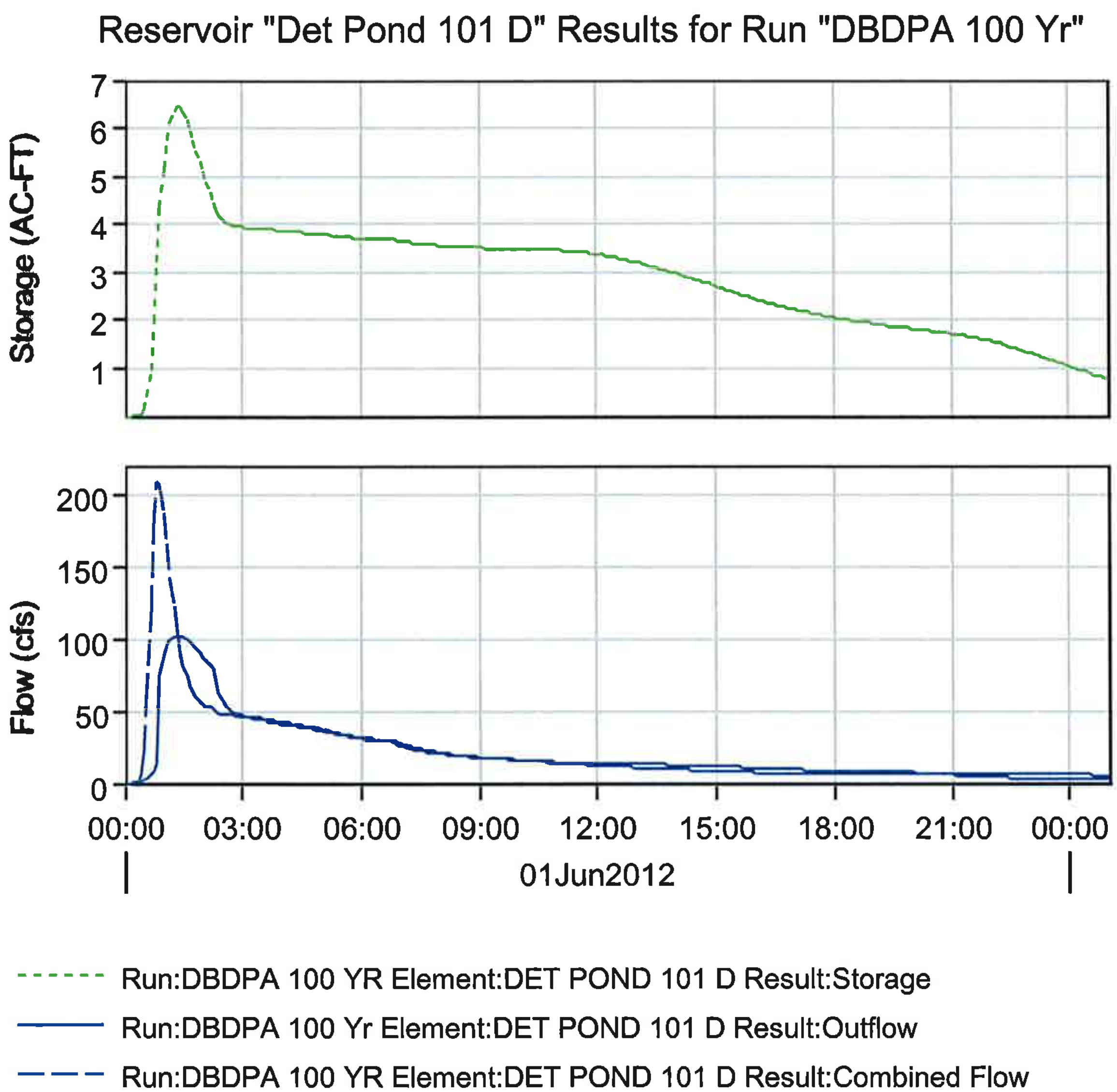


Basin 17F
80
70
60
50
40
30
20
10
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 17F Result:Direct Runoff


Basin 18F
Flow (cfs)
140
120
100
80
60
40
20
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:BASIN 18F Result:Direct Runoff

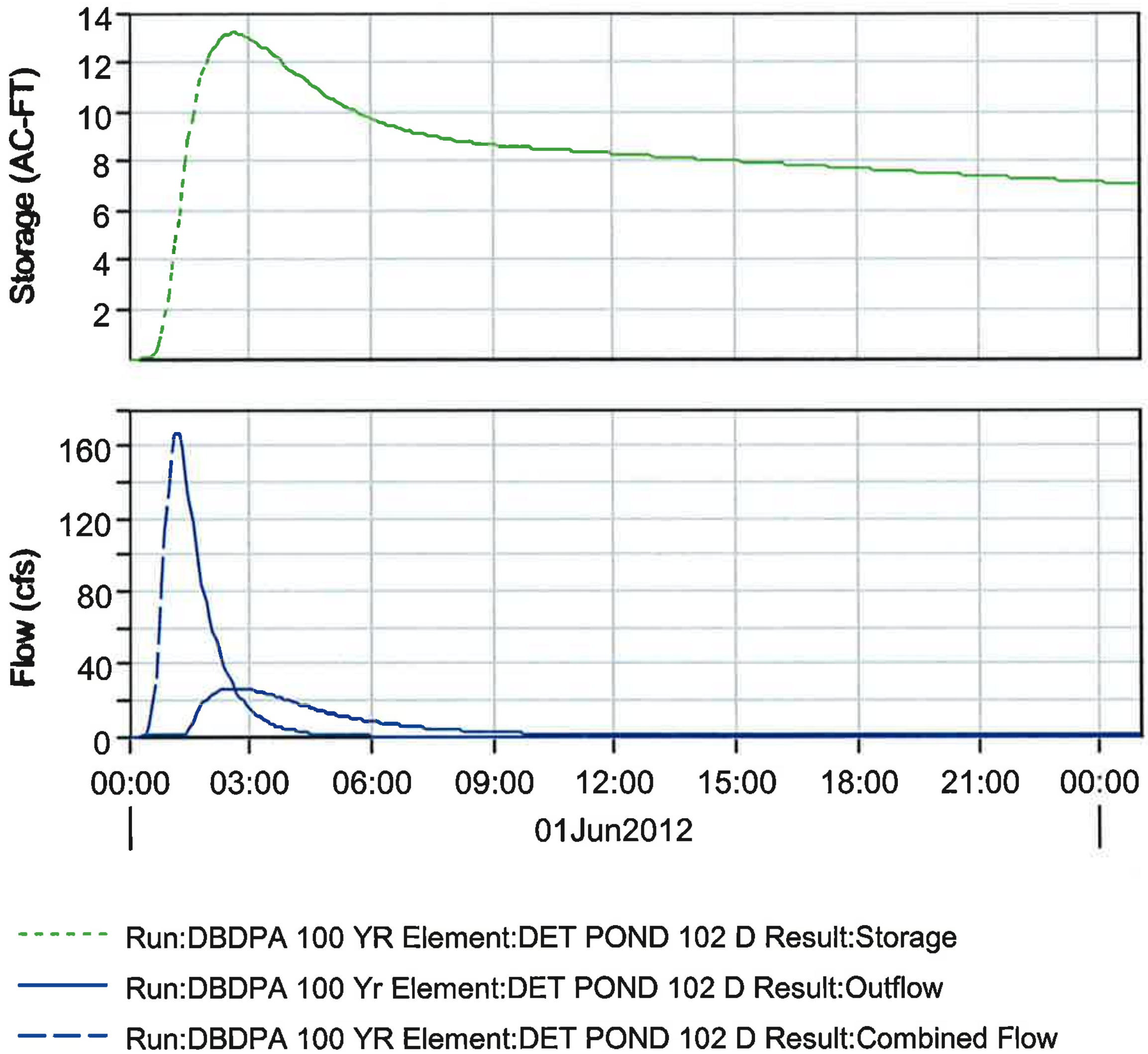
Reservoir "Det. Pond 100 D" Results for Run "DBDPA 100 Yr"

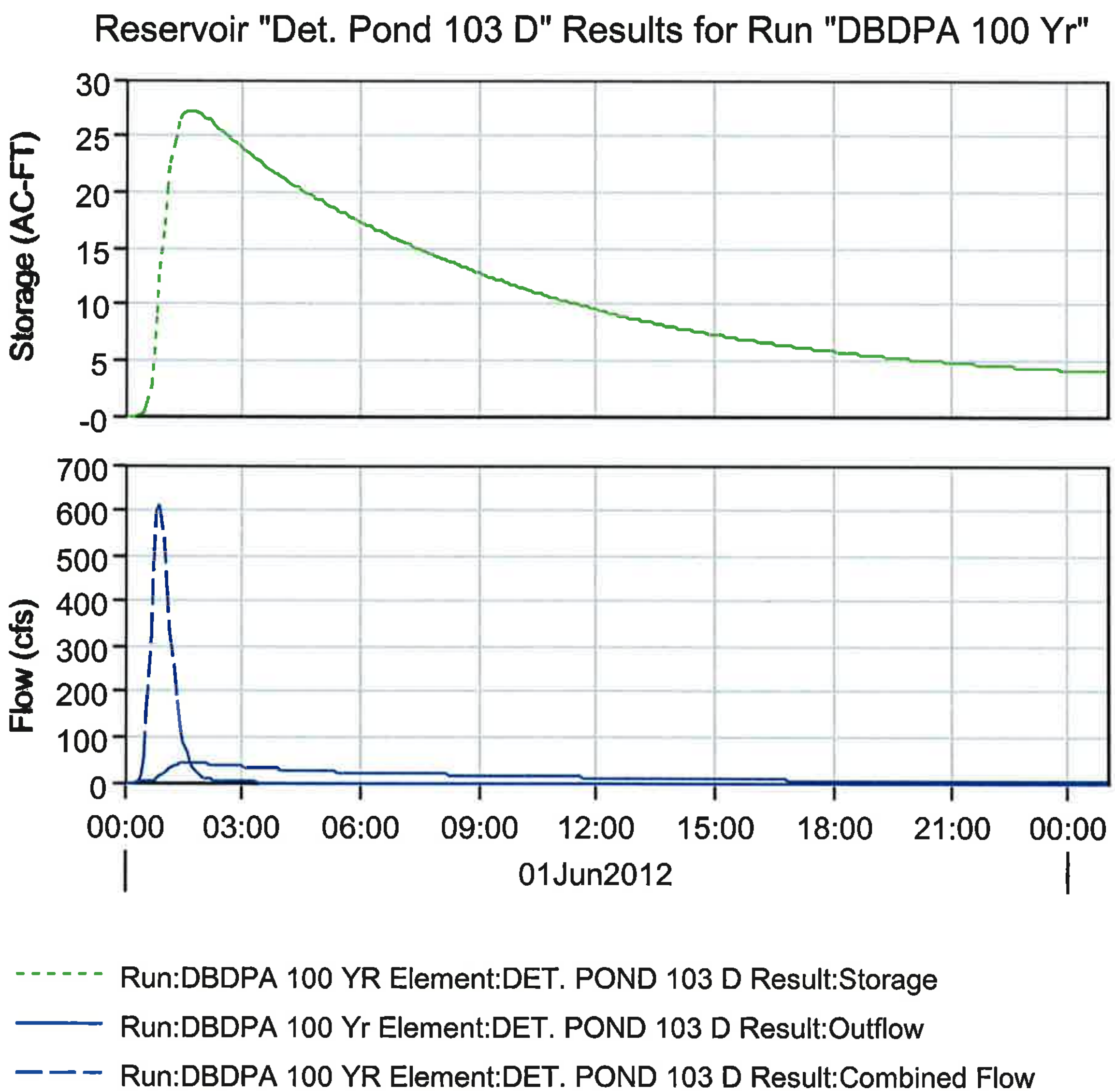




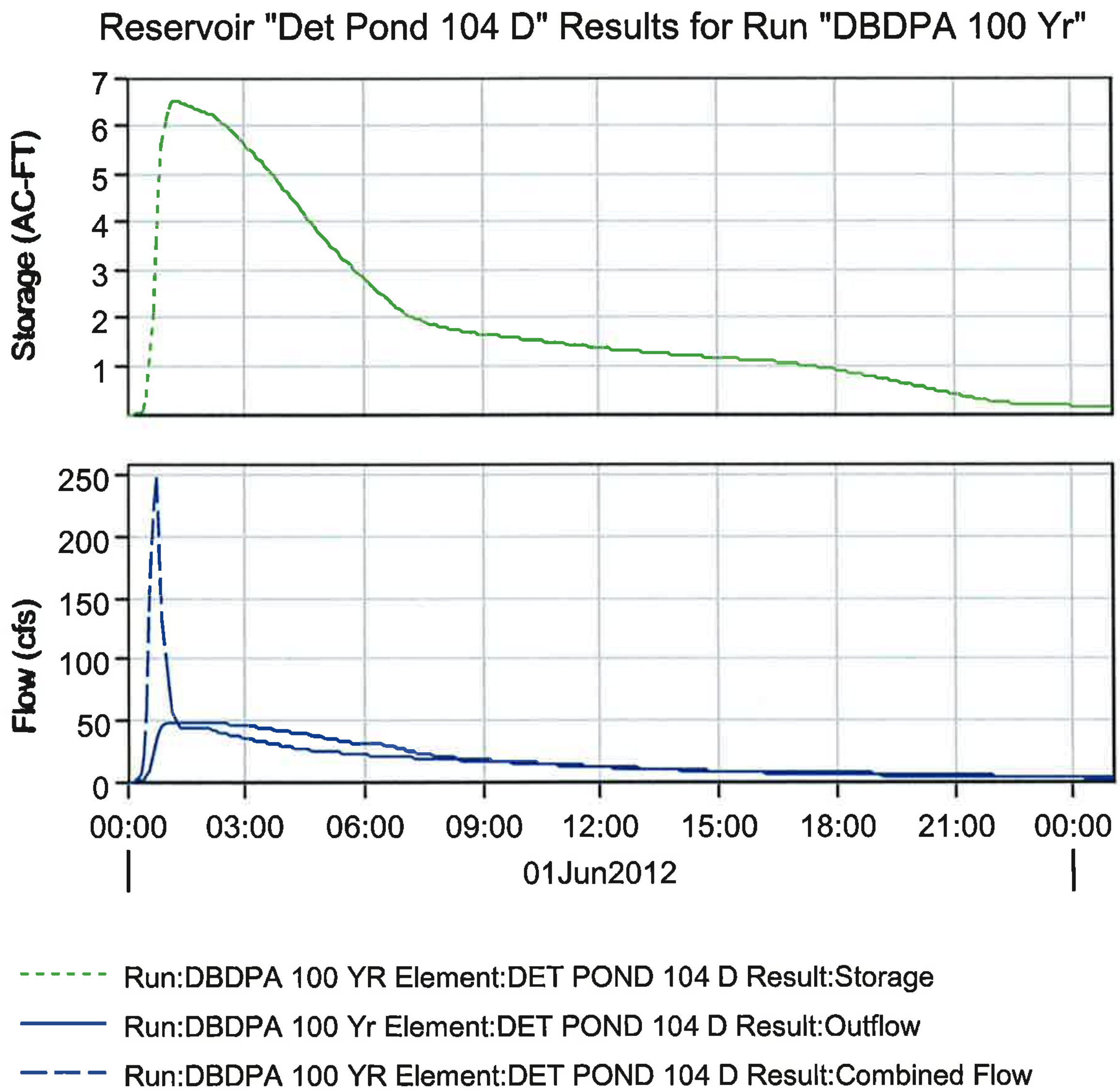
Reservoir "Det Pond 101 D" Results for Run "DBDPA 100 Yr"
Storage (AC-FT)
7
6
5
4
3
2
1
Flow (cfs)
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:DET POND 101 D Result:Storage
Run:DBDPA 100 Yr Element:DET POND 101 D Result:Outflow
Run:DBDPA 100 YR Element:DET POND 101 D Result:Combined Flow

Reservoir "Det Pond 102 D" Results for Run "DBDPA 100 Yr"

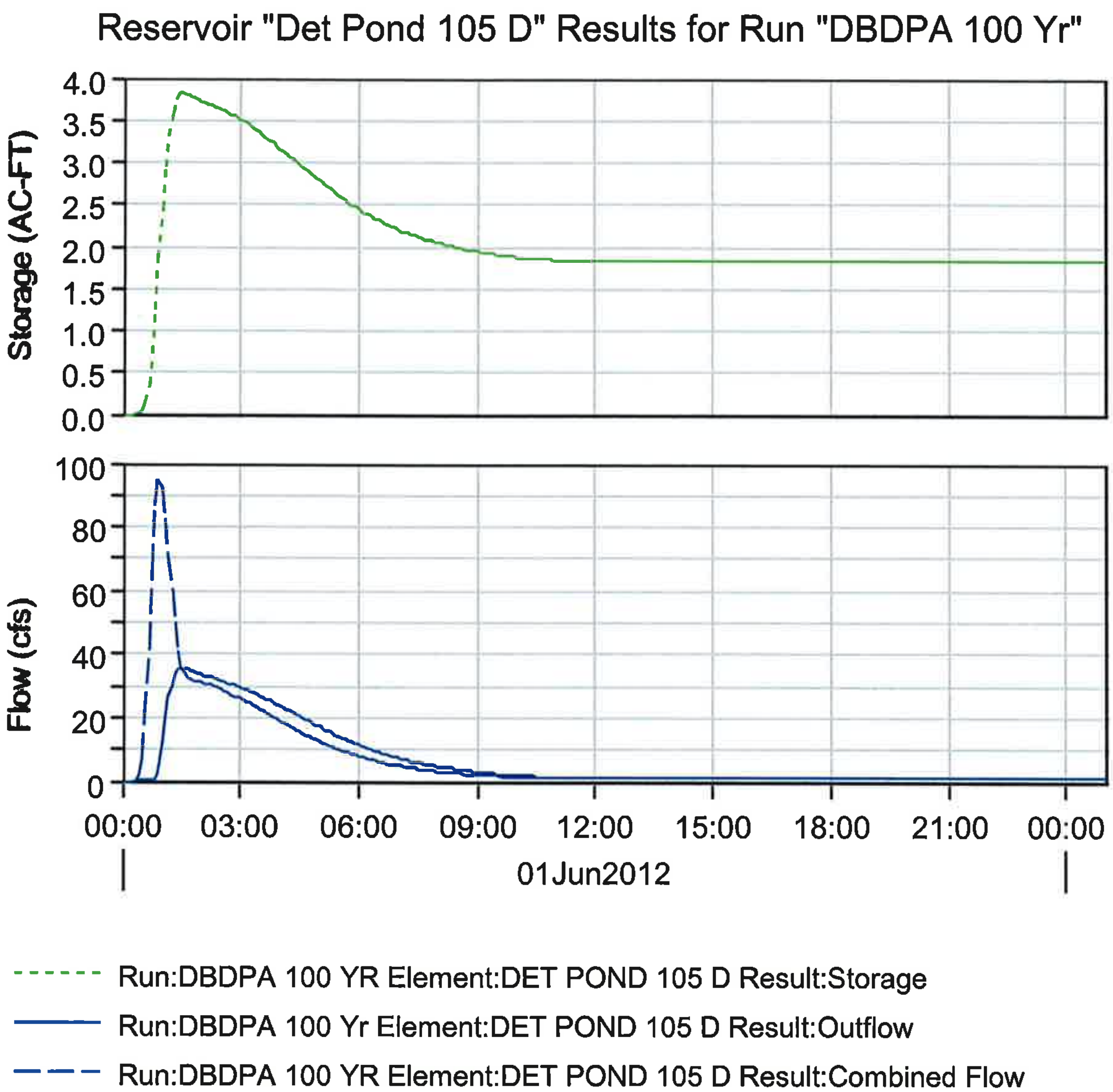




Reservoir "Det. Pond 103 D" Results for Run "DBDPA 100 Yr"
Storage (AC-FT)
30
25
20
15
10
5
-0
Flow (cfs)
700
600
500
400
300
200
100
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:DET. POND 103 D Result:Storage
Run:DBDPA 100 Yr Element:DET. POND 103 D Result:Outflow
Run:DBDPA 100 YR Element:DET. POND 103 D Result:Combined Flow

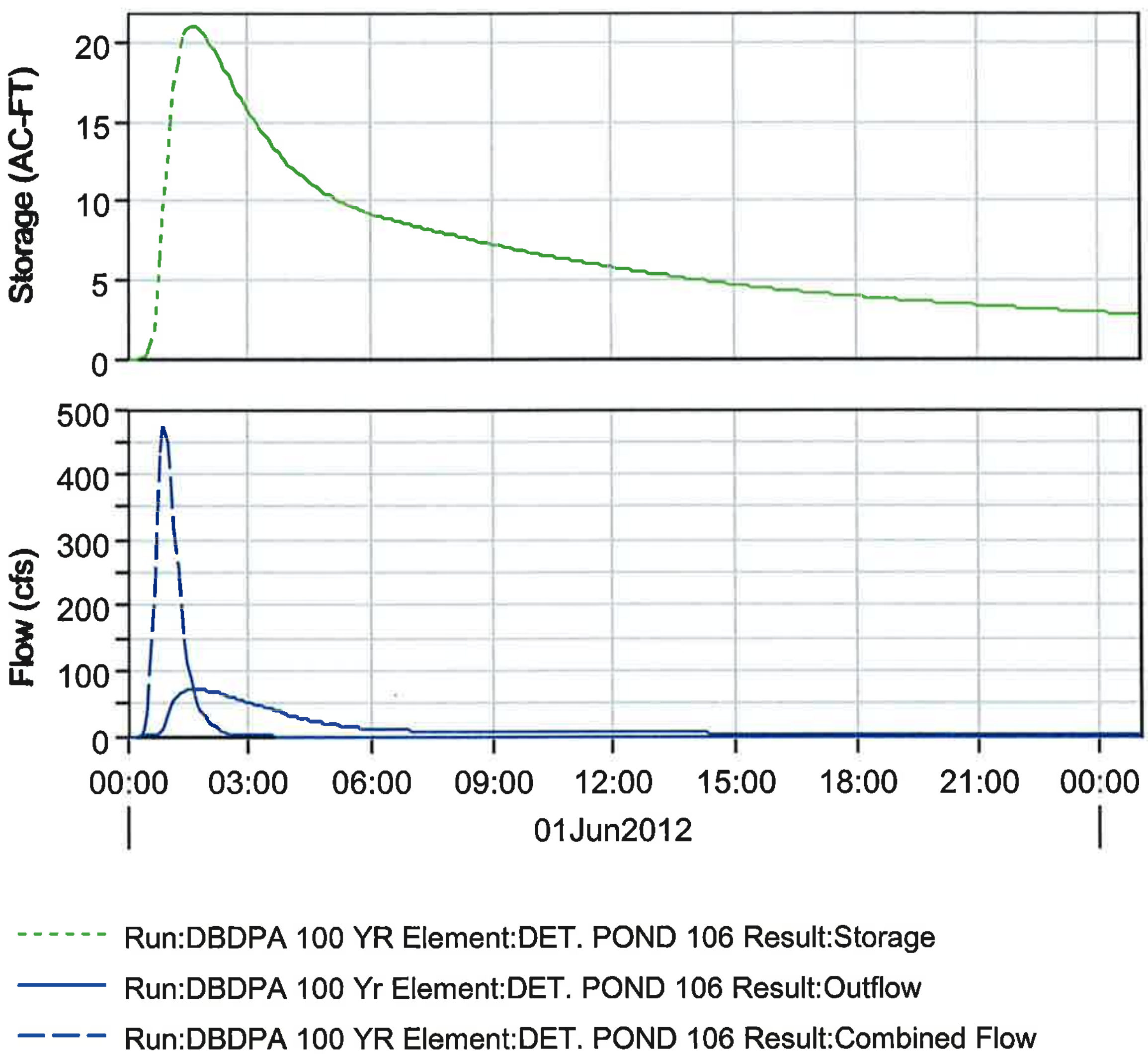


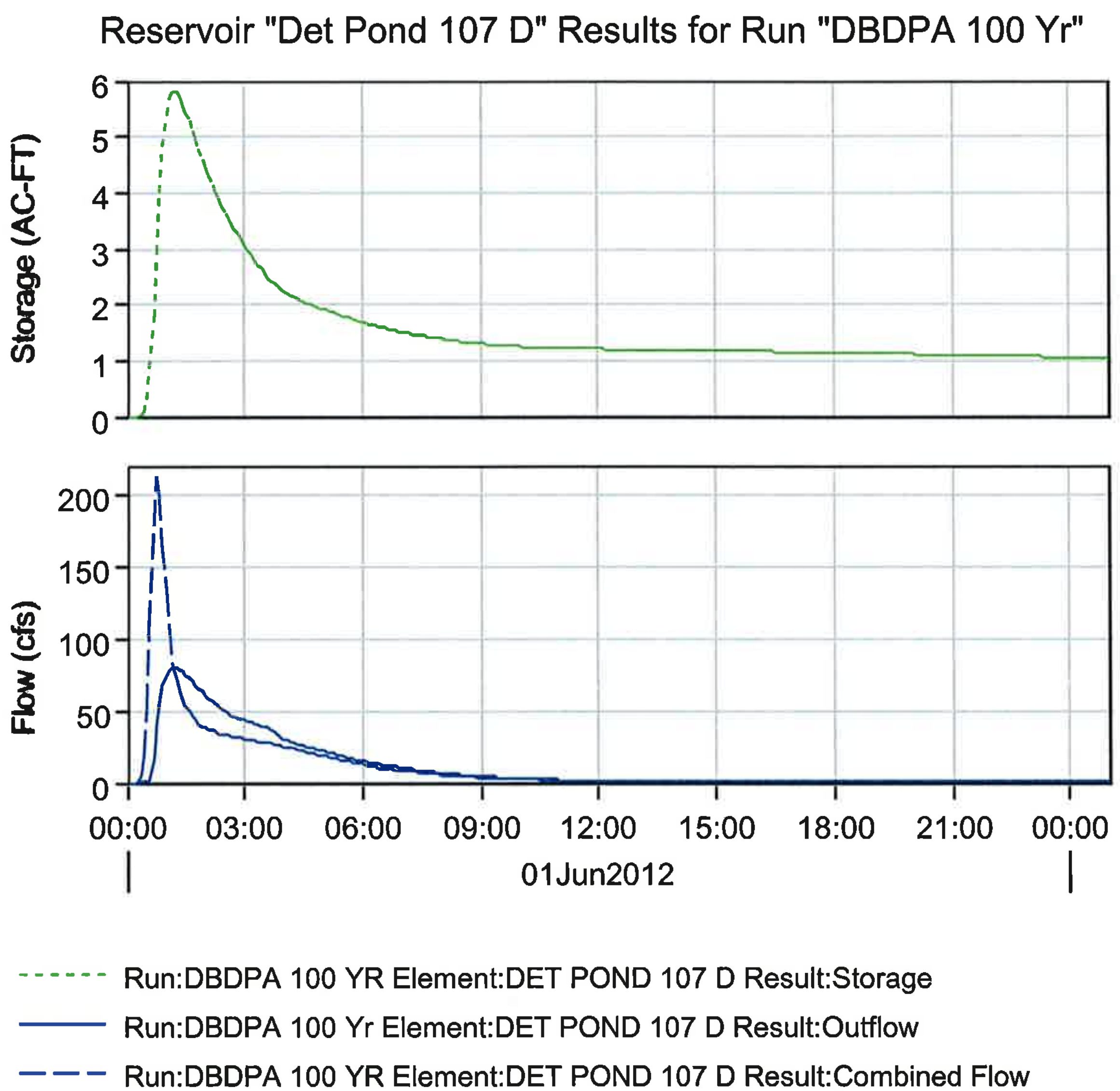
Reservoir "Det Pond 104 D" Results for Run "DBDPA 100 Yr"
Storage (AC-FT)
7
6
5
4
3
2
1
Flow (cfs)
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:DET POND 104 D Result:Storage
Run:DBDPA 100 Yr Element:DET POND 104 D Result:Outflow
Run:DBDPA 100 YR Element:DET POND 104 D Result:Combined Flow



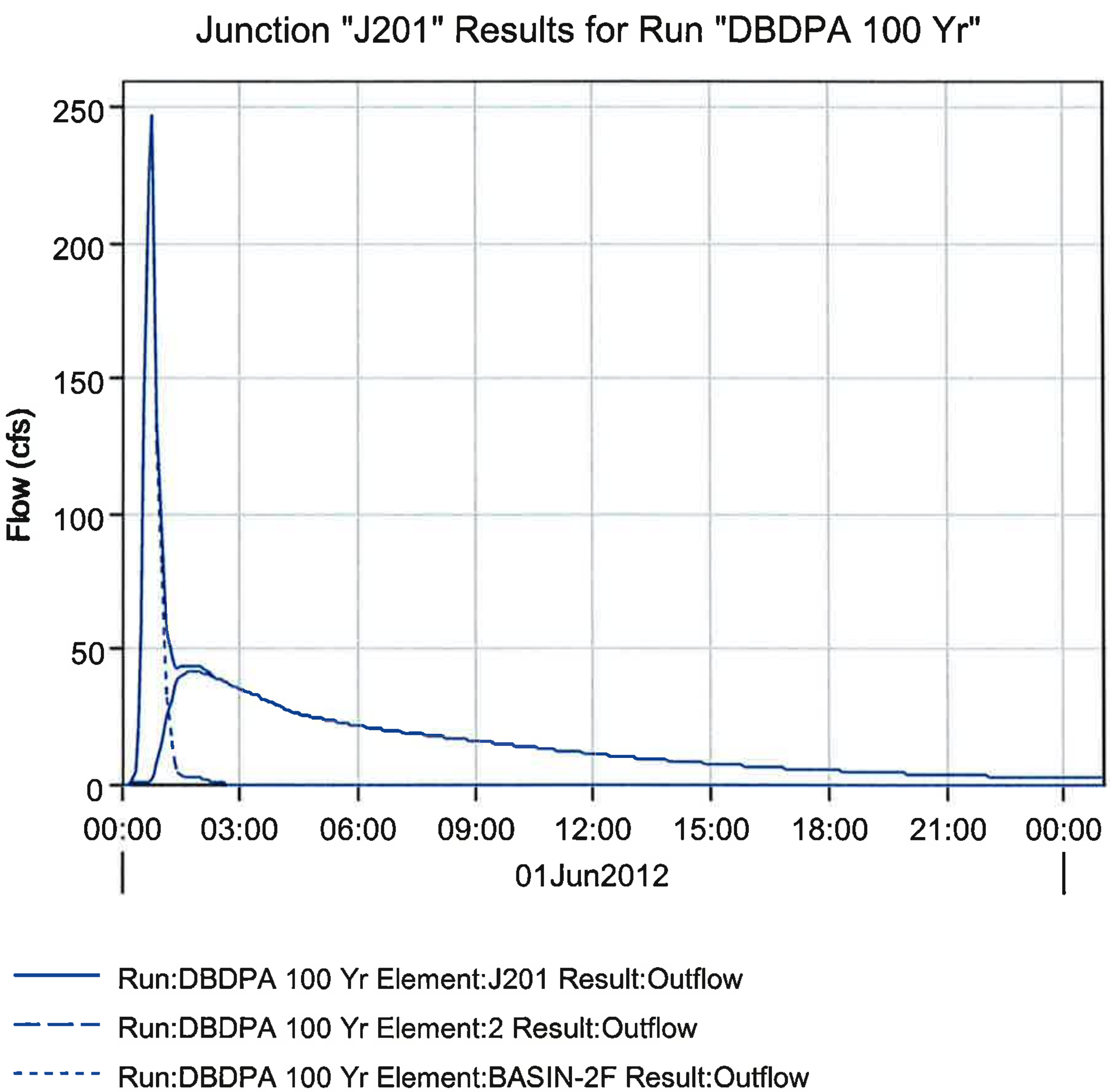
Reservoir "Det Pond 105 D" Results for Run "DBDPA 100 Yr"
Storage (AC-FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Flow (cfs)
100
80
60
40
20
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:DET POND 105 D Result:Storage
Run:DBDPA 100 Yr Element:DET POND 105 D Result:Outflow
Run:DBDPA 100 YR Element:DET POND 105 D Result:Combined Flow

Reservoir "Det. Pond 106" Results for Run "DBDPA 100 Yr"

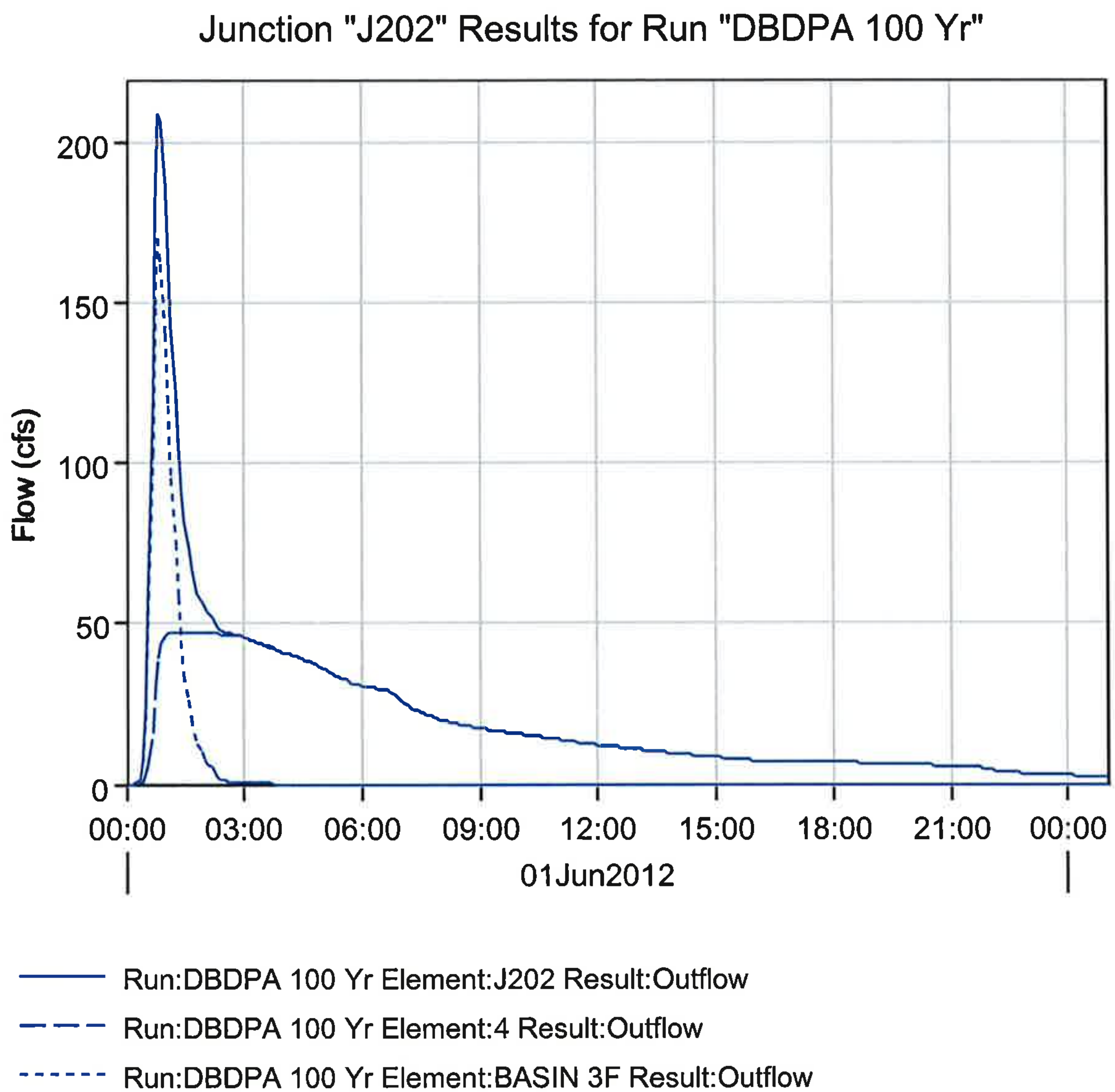




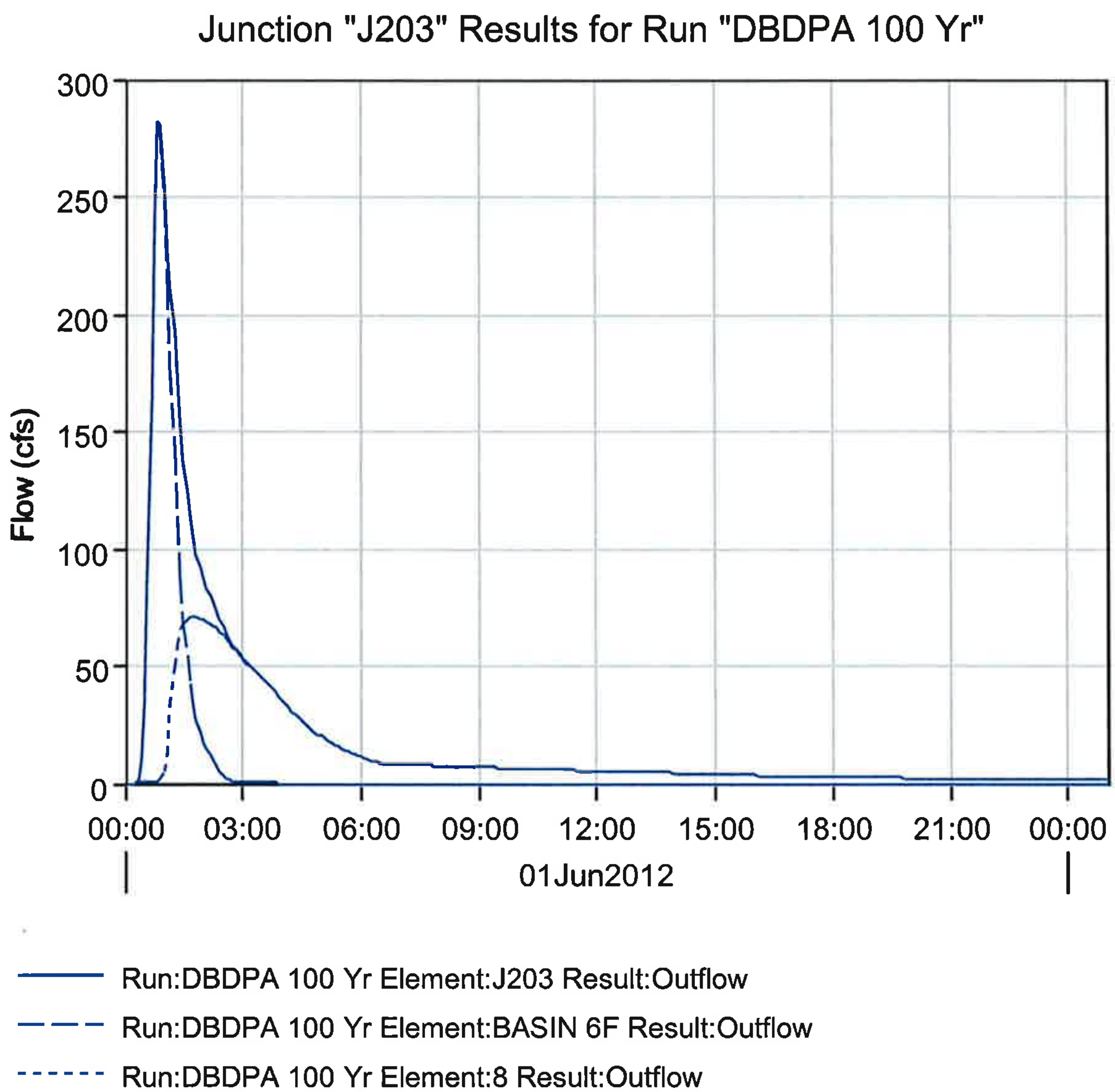
Reservoir "Det Pond 107 D" Results for Run "DBDPA 100 Yr"
Storage (AC-FT)
Flow (cfs)
0
1
2
3
4
5
6
0
50
100
150
200
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 YR Element:DET POND 107 D Result:Storage
Run:DBDPA 100 Yr Element:DET POND 107 D Result:Outflow
Run:DBDPA 100 YR Element:DET POND 107 D Result:Combined Flow



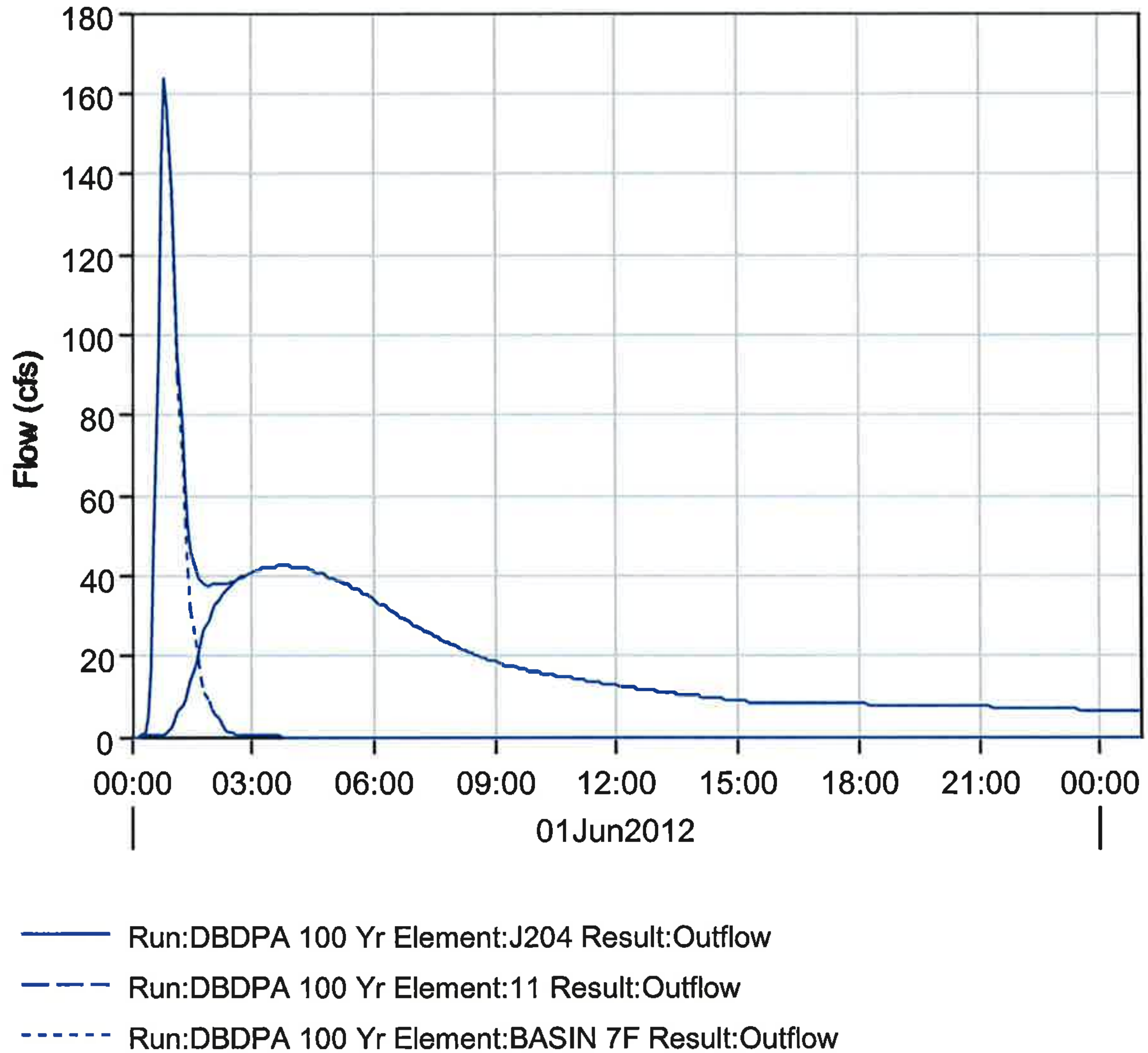
Junction "J201" Results for Run "DBDPA 100 Yr"
Flow (cfs)
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J201 Result:Outflow
Run:DBDPA 100 Yr Element:2 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN-2F Result:Outflow

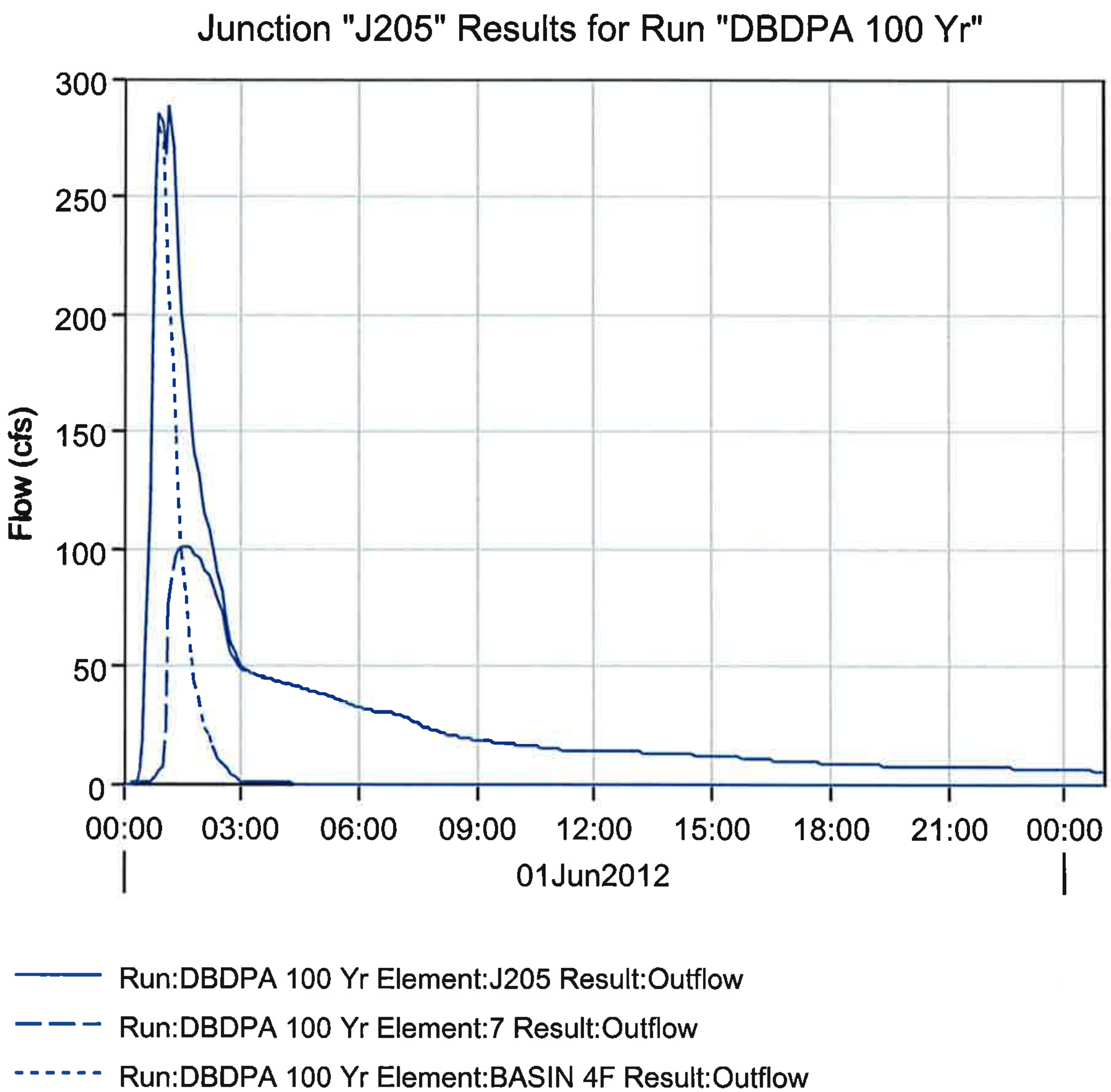


Junction "J202" Results for Run "DBDPA 100 Yr"
Flow (cfs)
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J202 Result:Outflow
Run:DBDPA 100 Yr Element:4 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 3F Result:Outflow

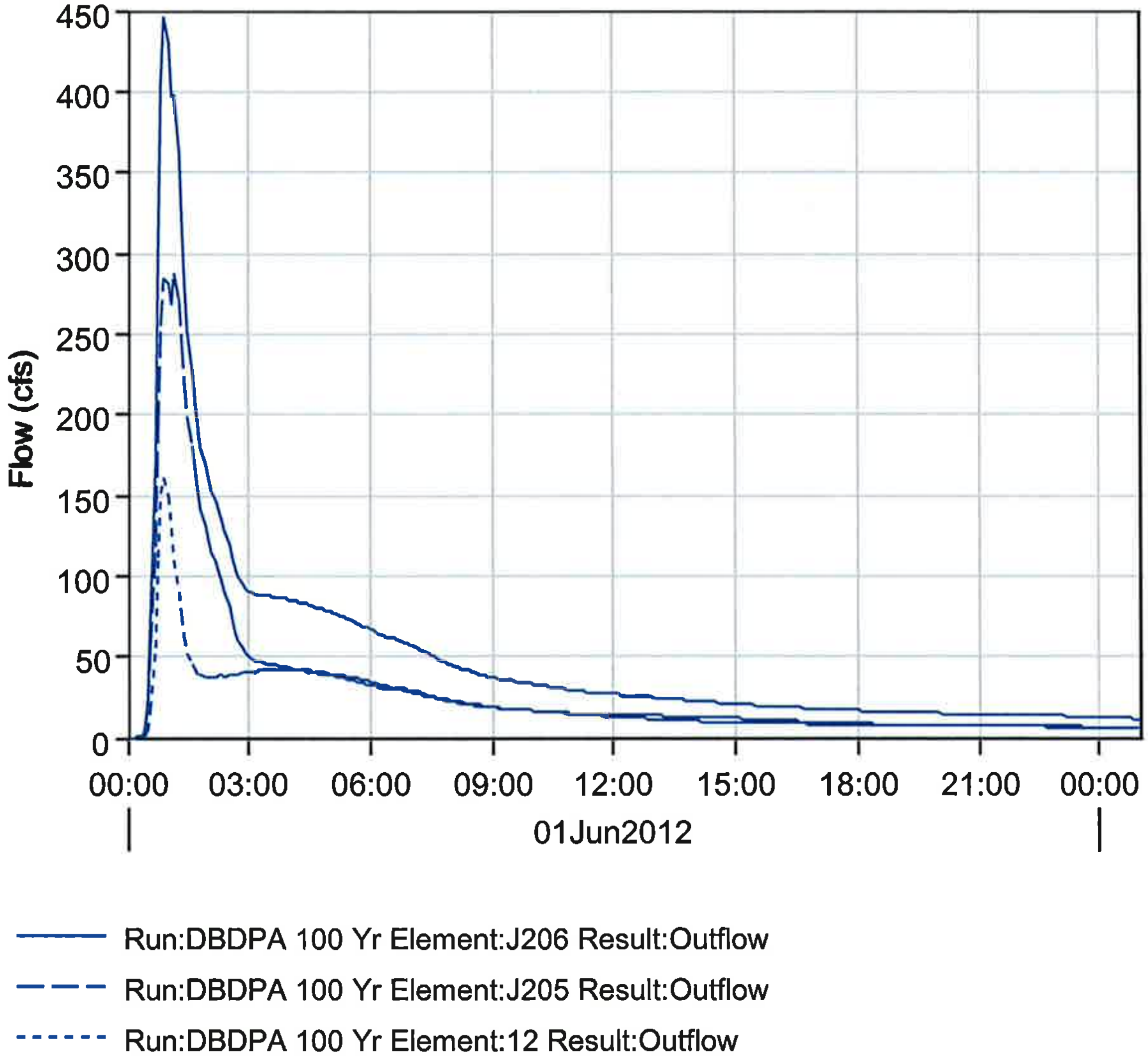

Junction "J203" Results for Run "DBDPA 100 Yr"
300
250
200
150
100
50
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J203 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 6F Result:Outflow
Run:DBDPA 100 Yr Element:8 Result:Outflow

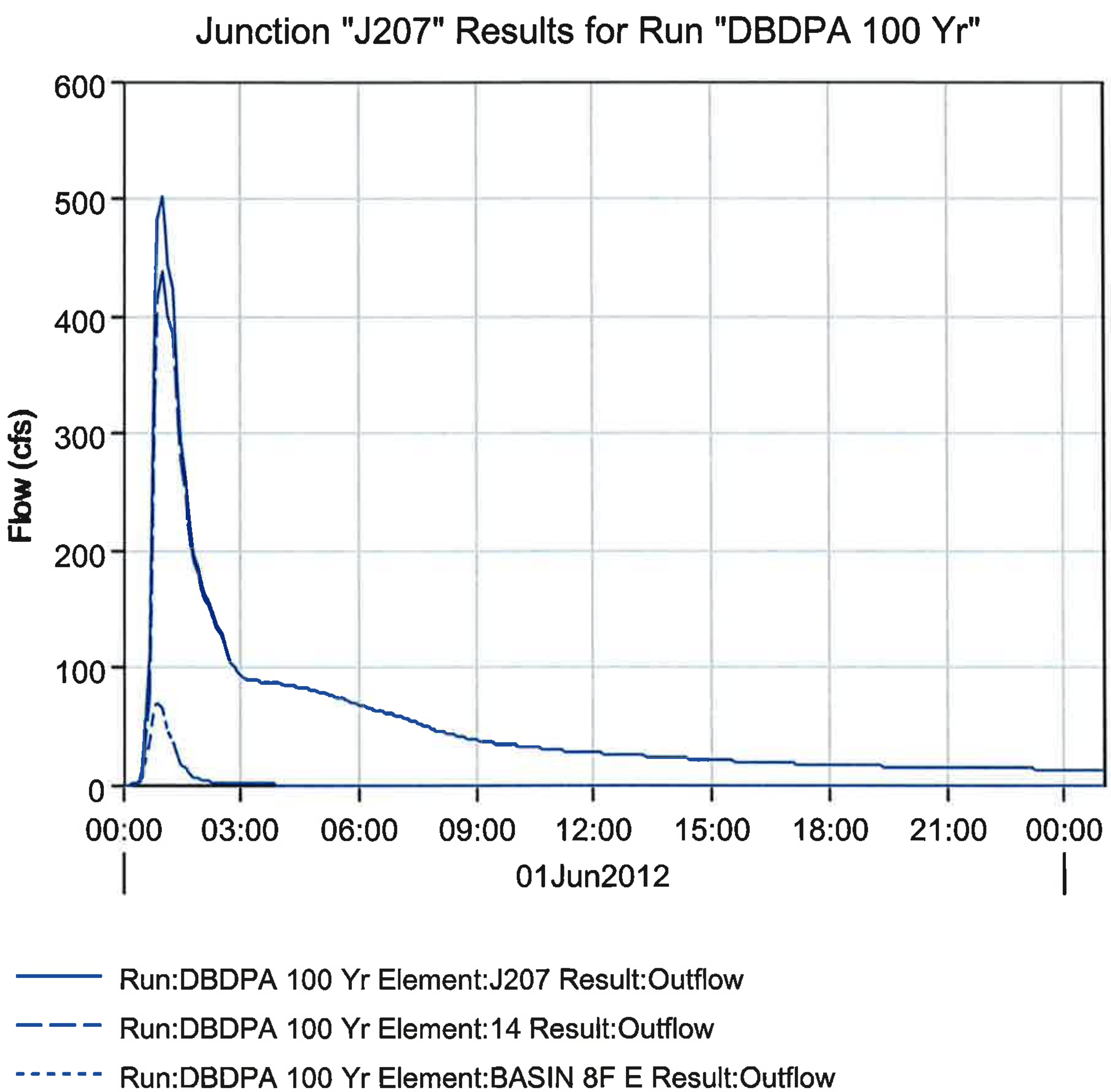
Junction "J204" Results for Run "DBDPA 100 Yr"

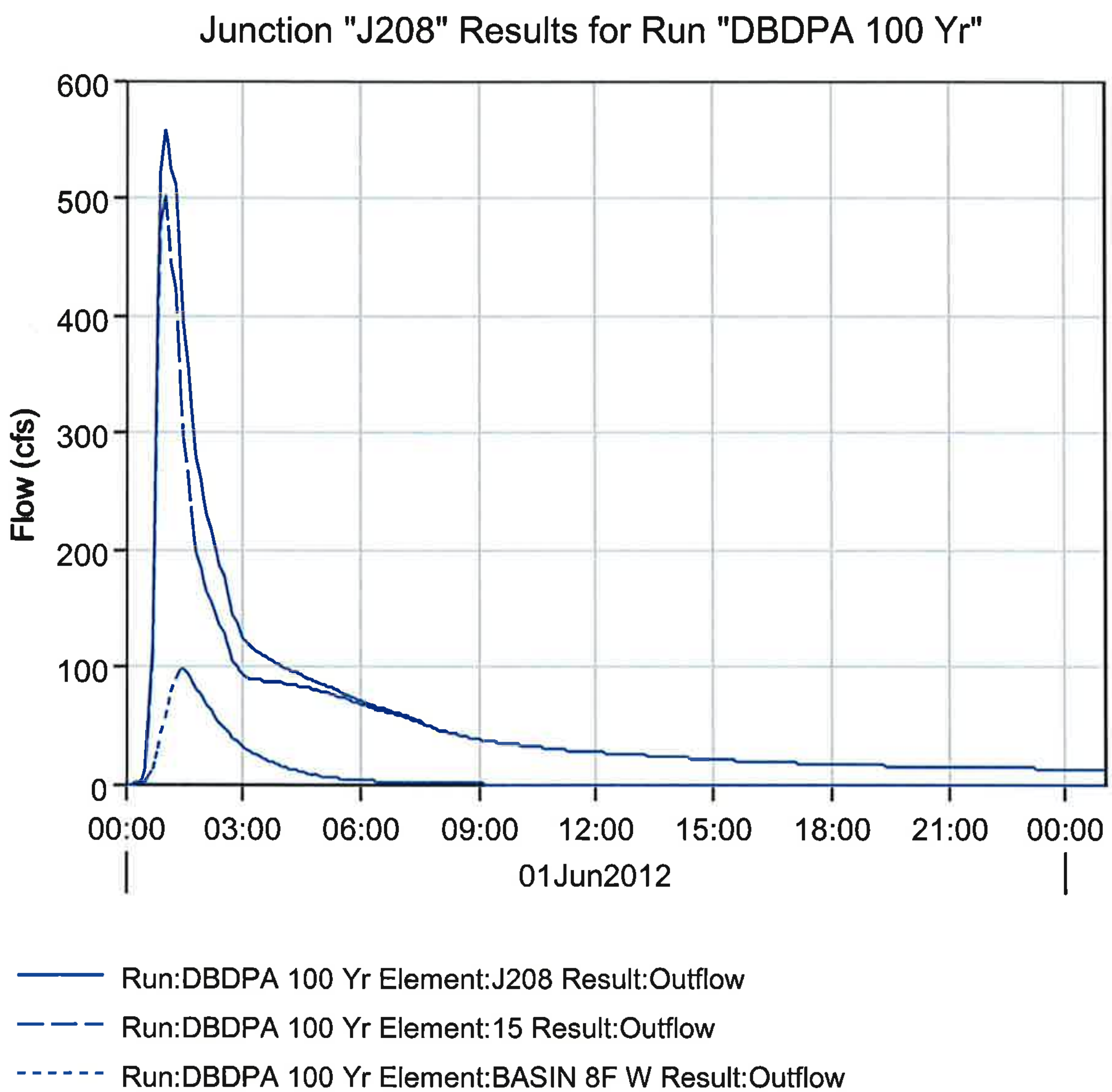


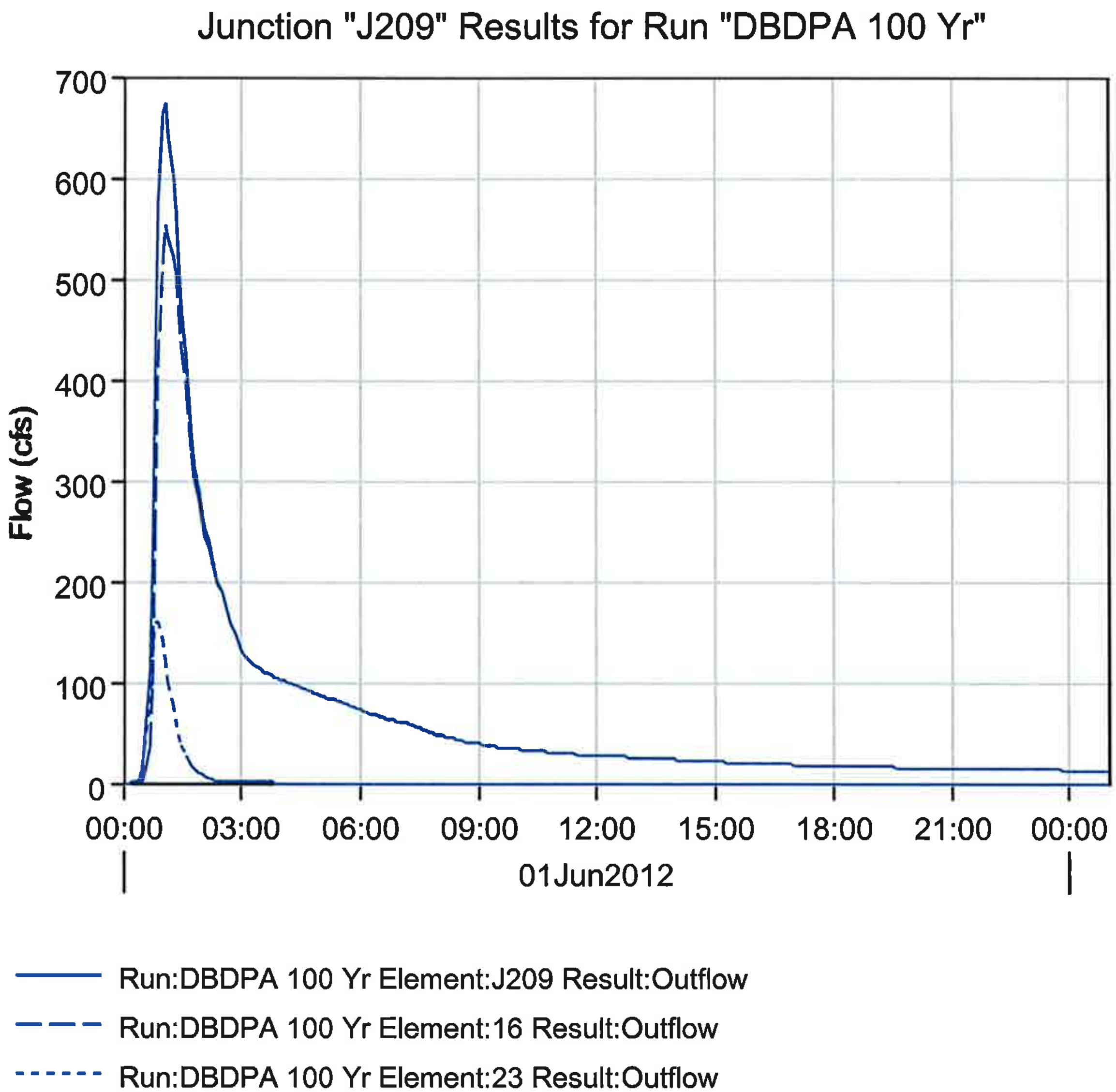

Junction "J205" Results for Run "DBDPA 100 Yr"
Flow (cfs)
300
250
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J205 Result:Outflow
Run:DBDPA 100 Yr Element:7 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 4F Result:Outflow

Junction "J206" Results for Run "DBDPA 100 Yr"

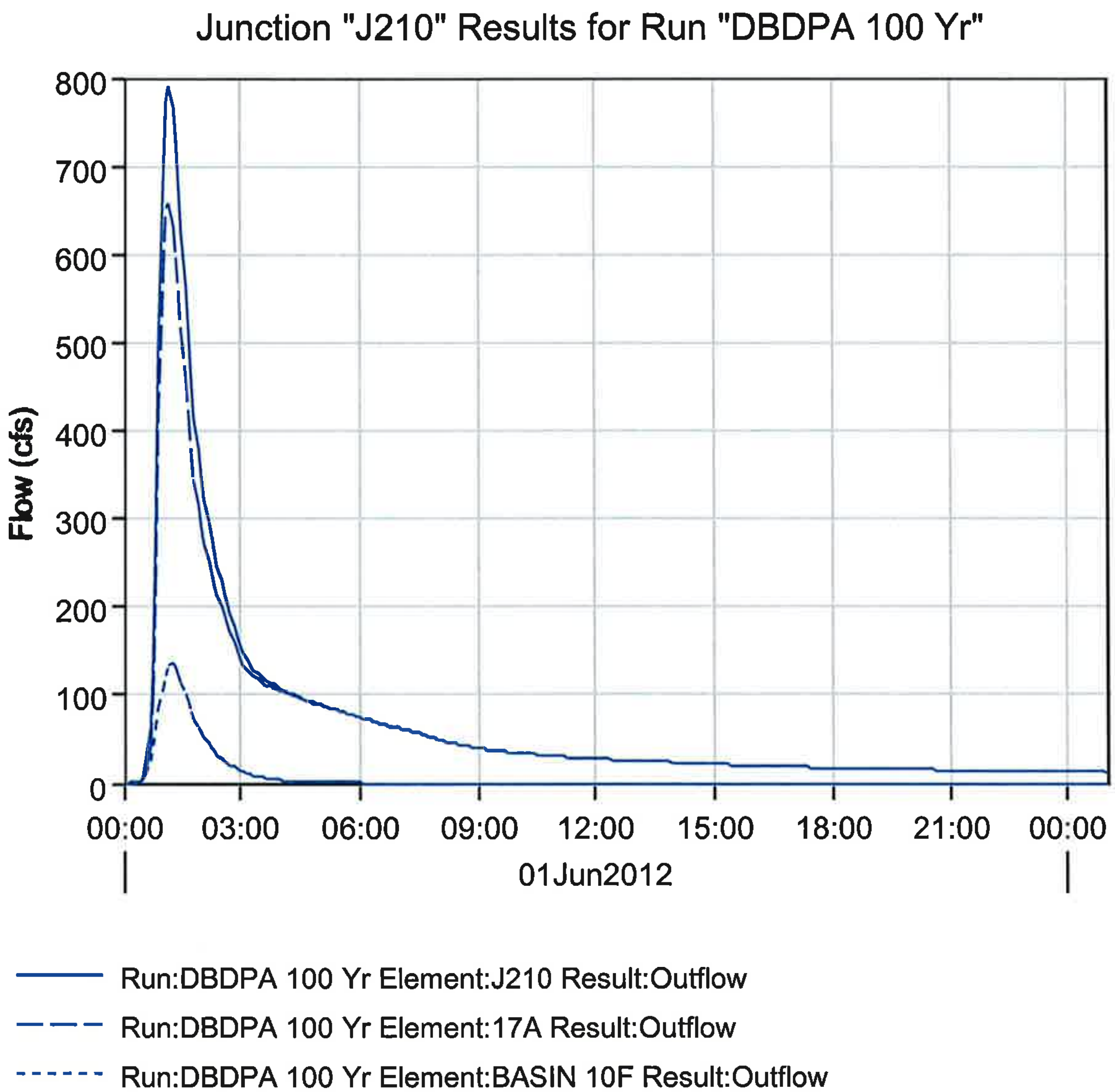



Junction "J207" Results for Run "DBDPA 100 Yr"
600
500
400
300
200
100
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J207 Result:Outflow
Run:DBDPA 100 Yr Element:14 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 8F E Result:Outflow

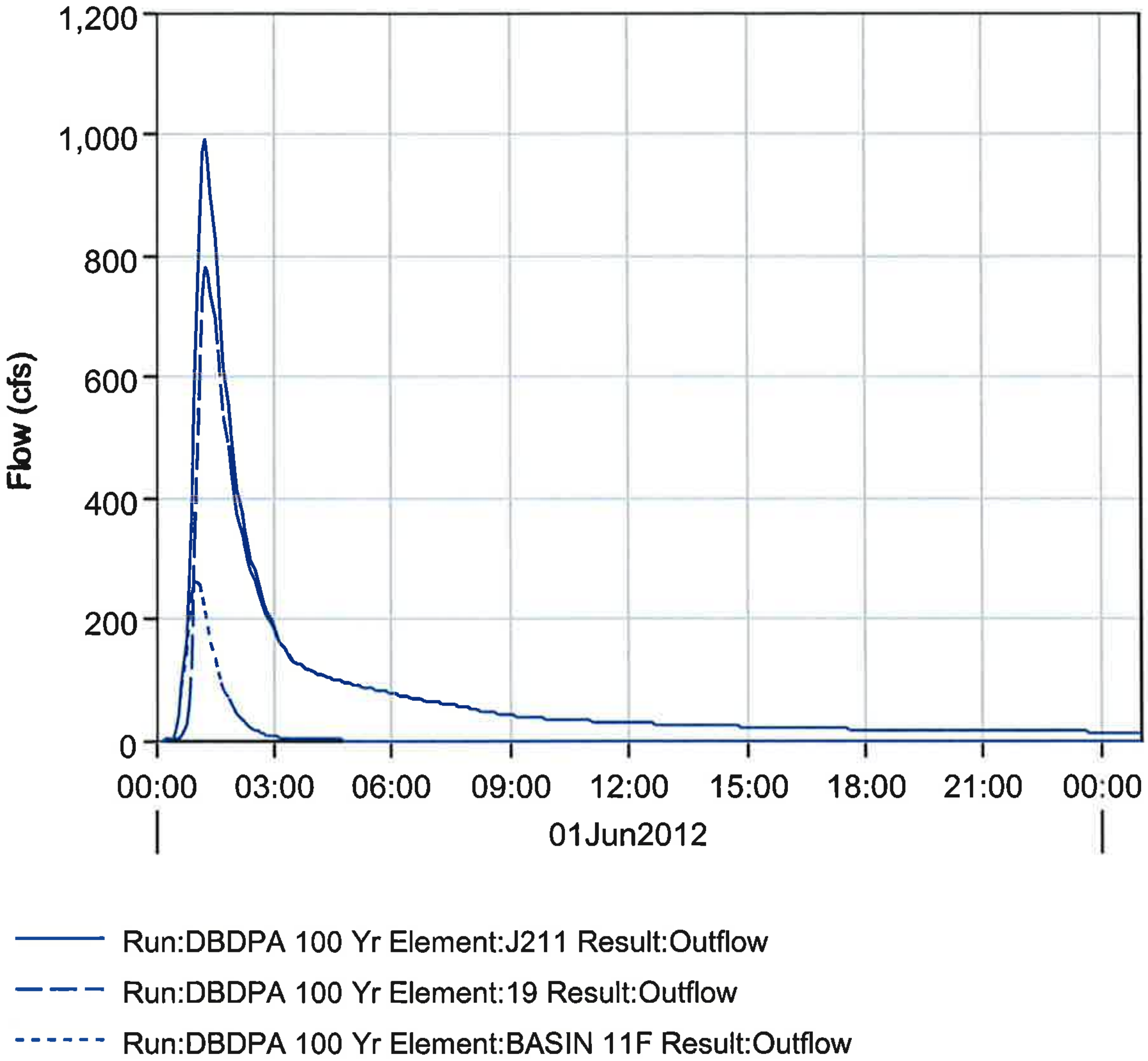

Junction "J208" Results for Run "DBDPA 100 Yr"
Flow (cfs)
600
500
400
300
200
100
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J208 Result:Outflow
Run:DBDPA 100 Yr Element:15 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 8F W Result:Outflow

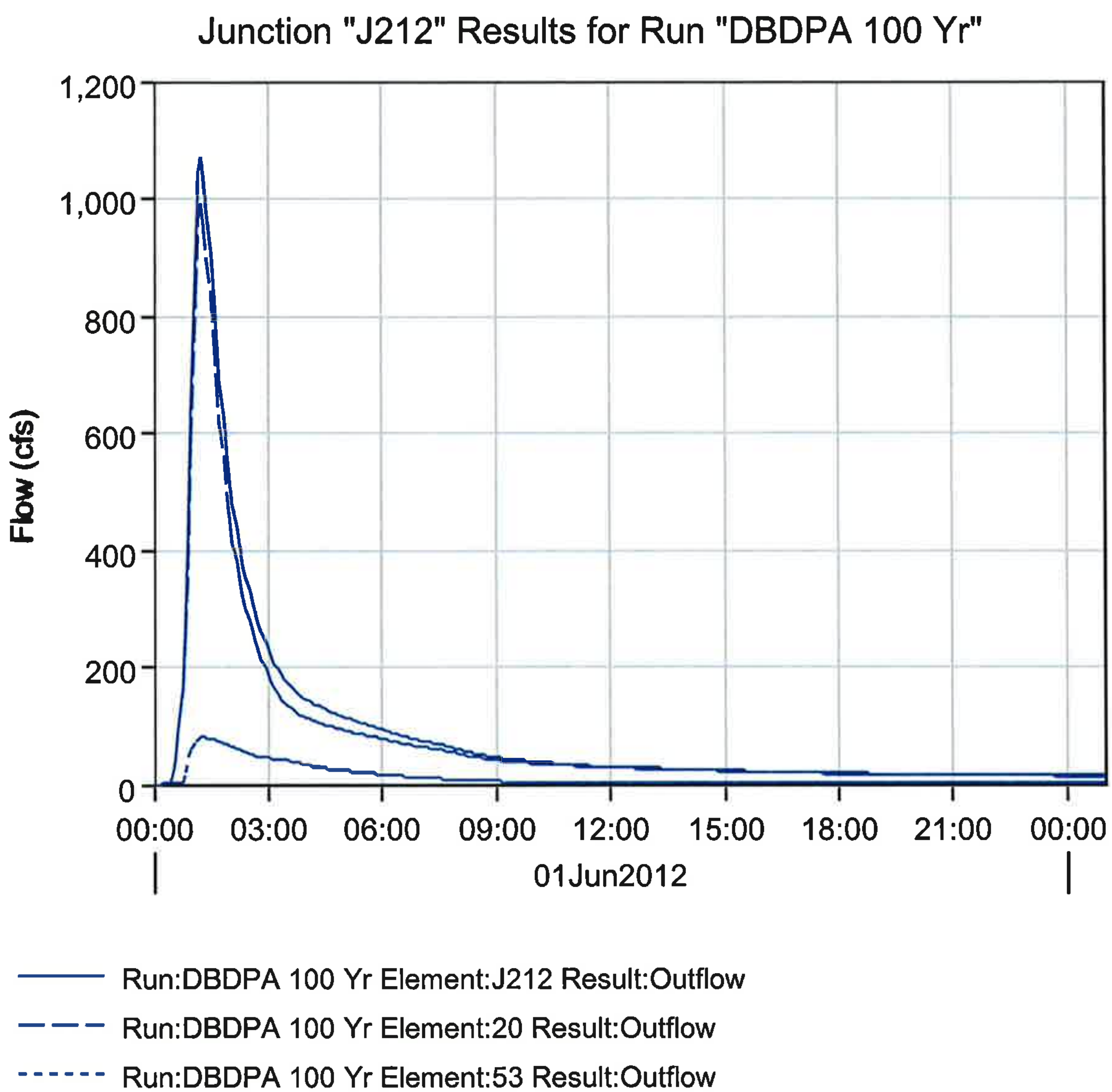


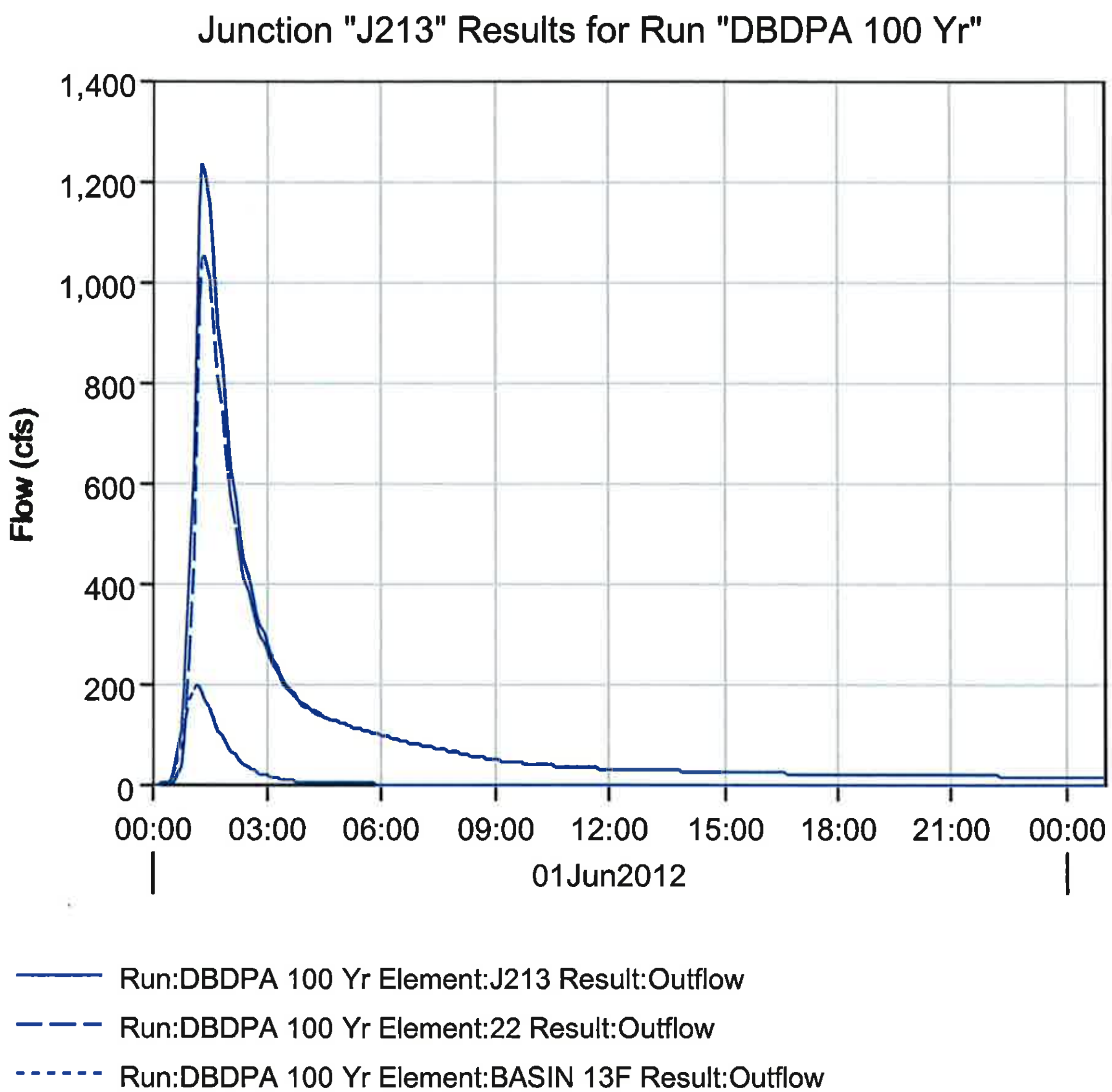
Junction "J209" Results for Run "DBDPA 100 Yr"
700
600
500
400
300
200
100
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J209 Result:Outflow
Run:DBDPA 100 Yr Element:16 Result:Outflow
Run:DBDPA 100 Yr Element:23 Result:Outflow


Junction "J210" Results for Run "DBDPA 100 Yr"
Flow (cfs)
800
700
600
500
400
300
200
100
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J210 Result:Outflow
Run:DBDPA 100 Yr Element:17A Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 10F Result:Outflow

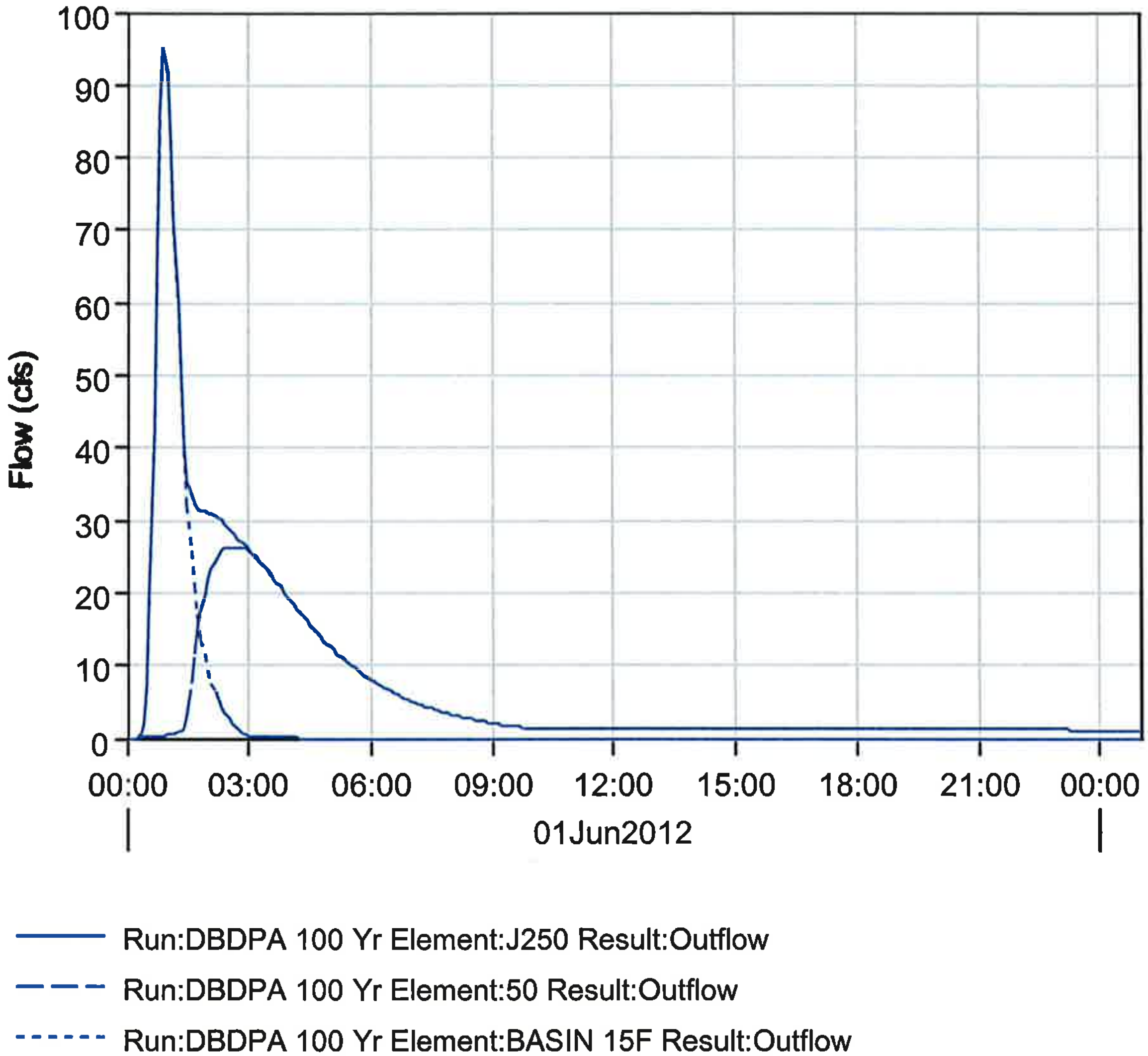
Junction "J211" Results for Run "DBDPA 100 Yr"

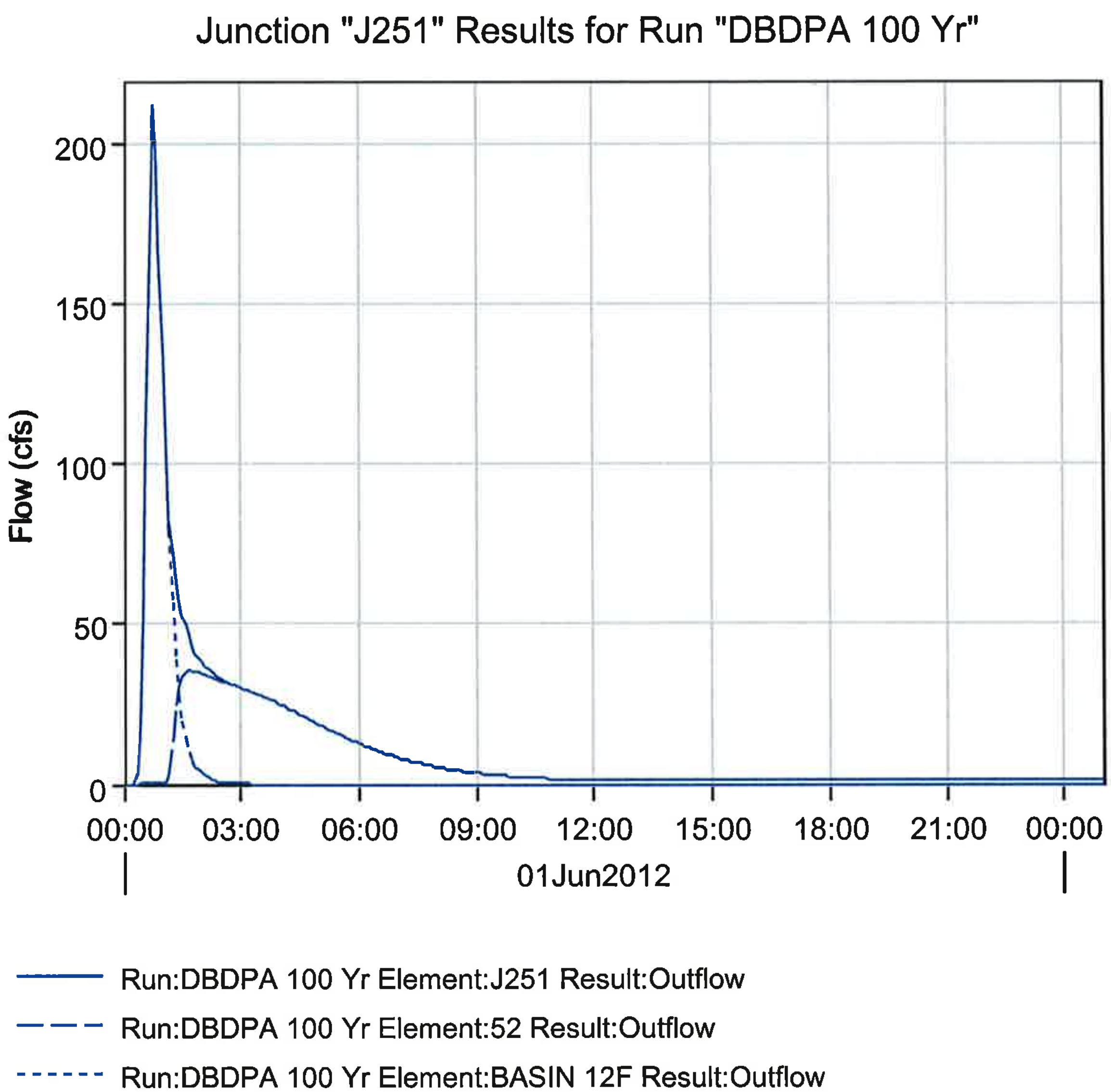


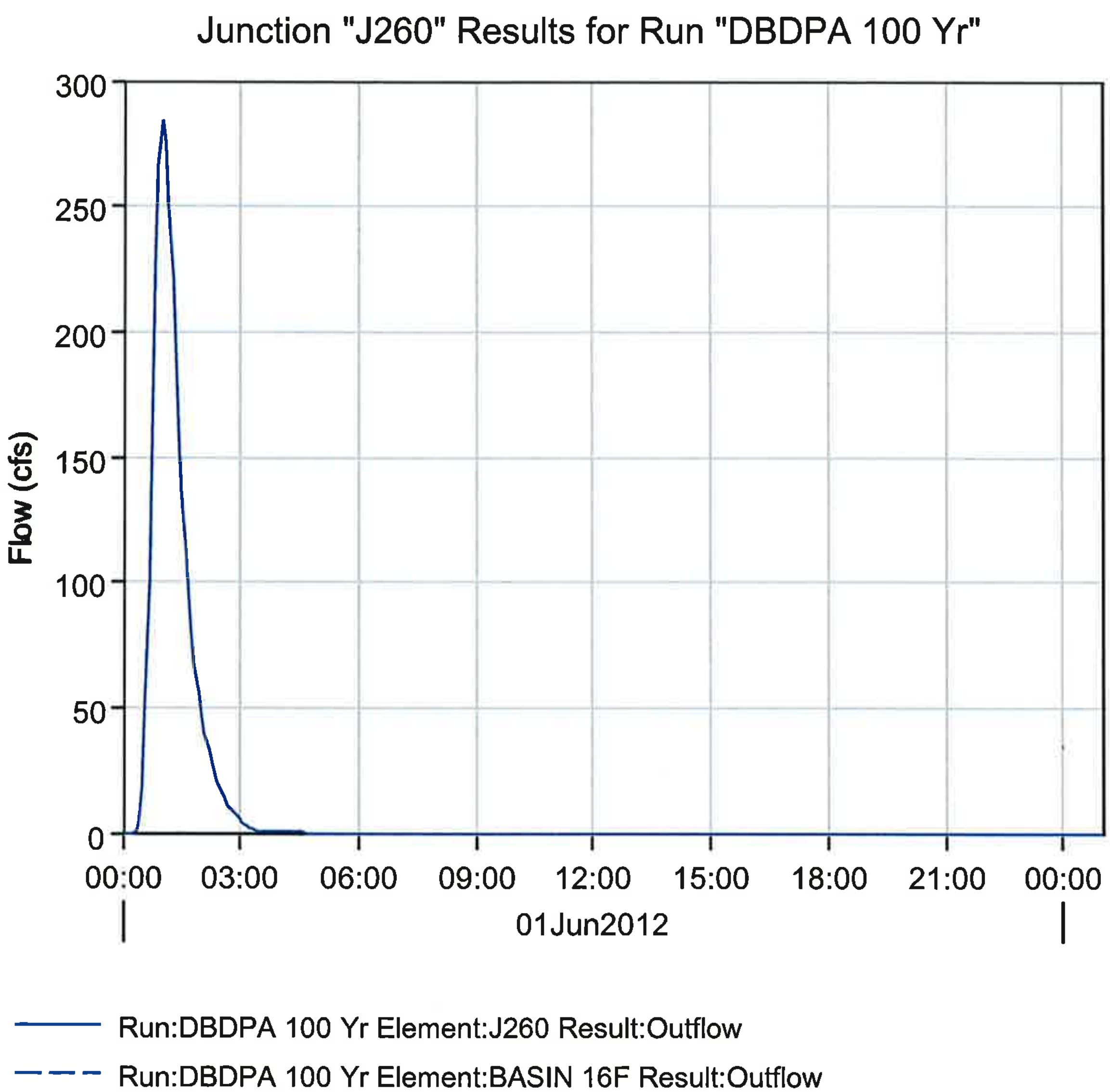

Junction "J212" Results for Run "DBDPA 100 Yr"
Flow (cfs)
1,200
1,000
800
600
400
200
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J212 Result:Outflow
Run:DBDPA 100 Yr Element:20 Result:Outflow
Run:DBDPA 100 Yr Element:53 Result:Outflow


Junction "J213" Results for Run "DBDPA 100 Yr"
Flow (cfs)
1,400
1,200
1,000
800
600
400
200
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J213 Result:Outflow
Run:DBDPA 100 Yr Element:22 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 13F Result:Outflow

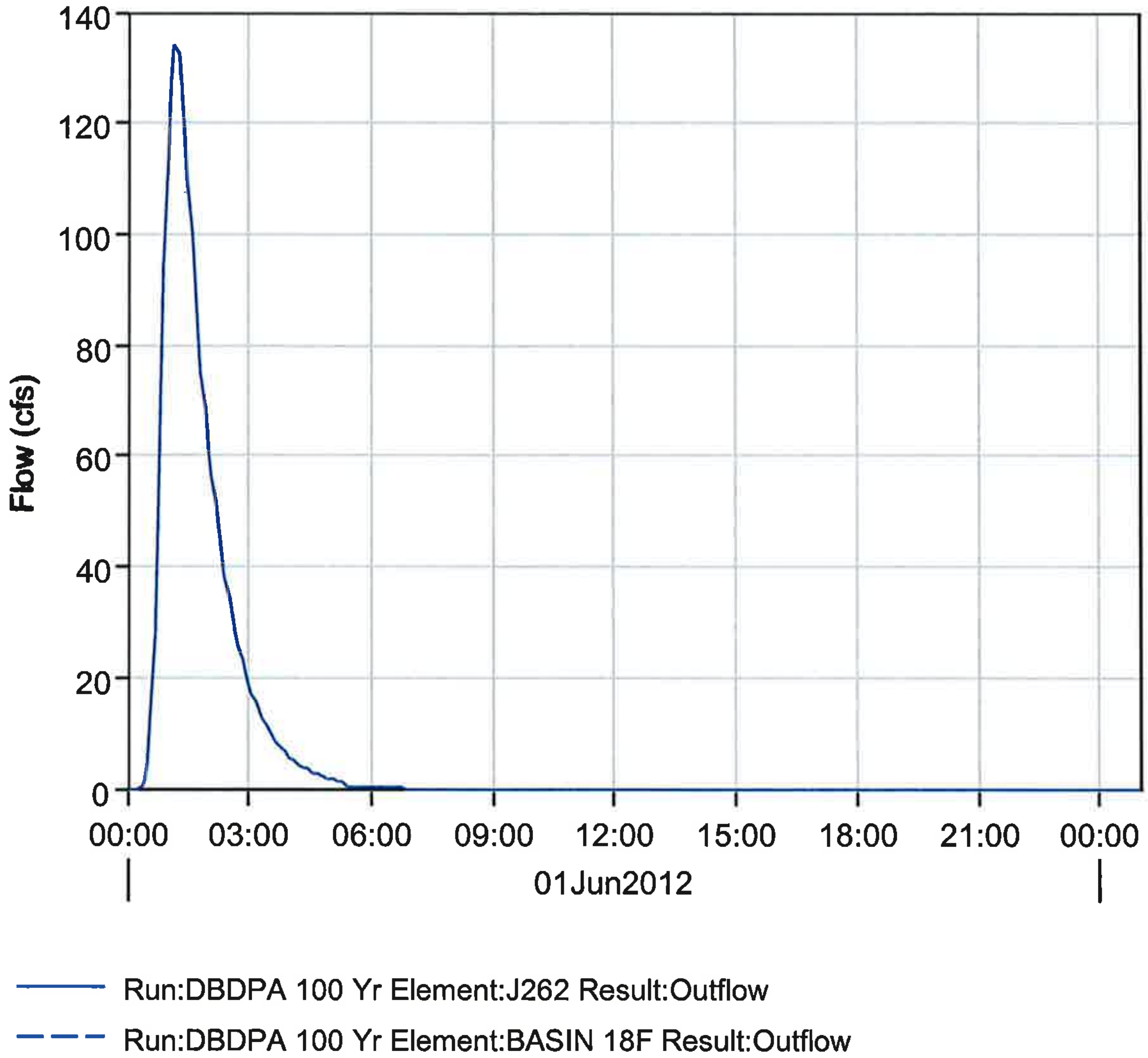
Junction "J250" Results for Run "DBDPA 100 Yr"

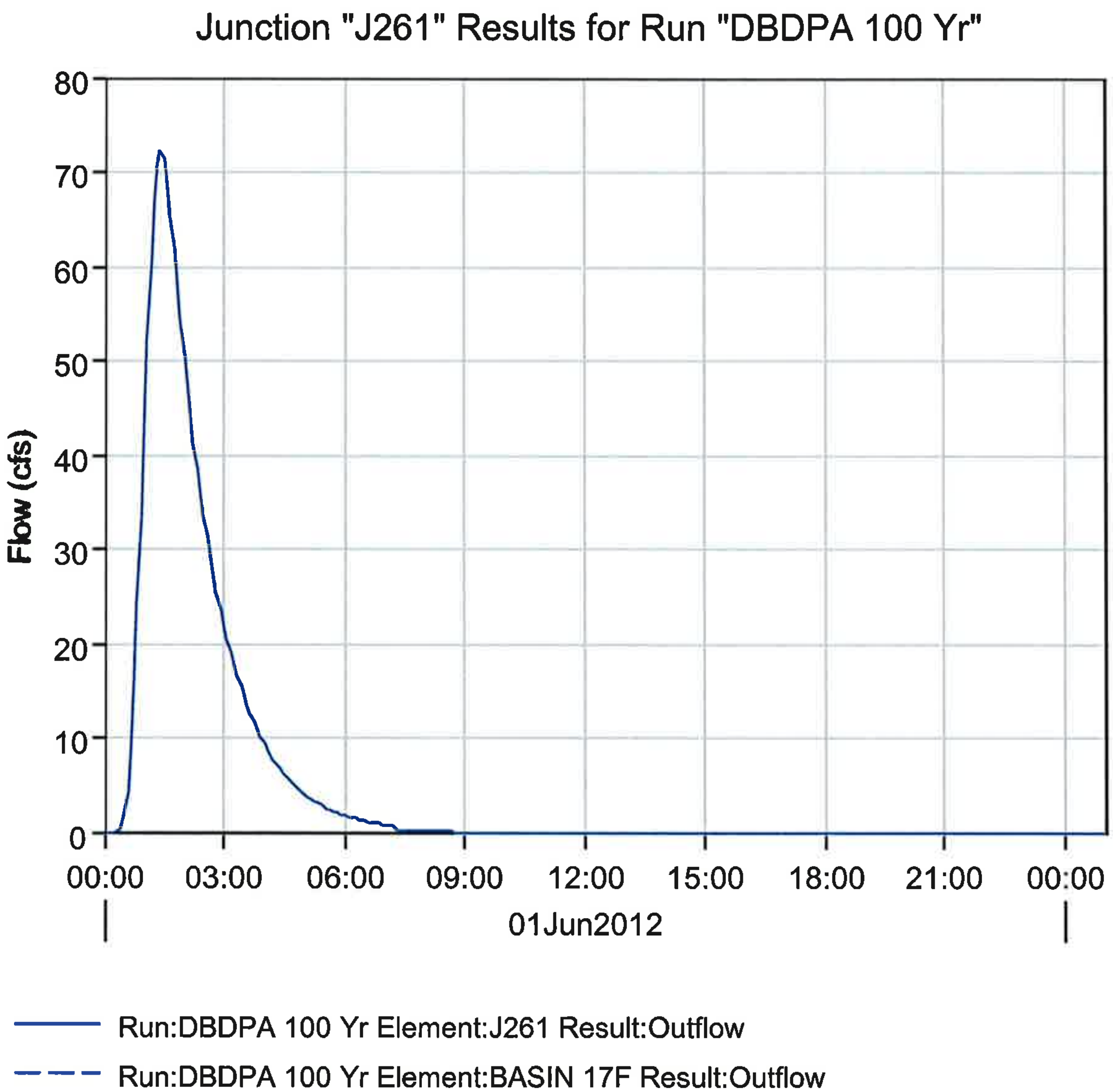



Junction "J251" Results for Run "DBDPA 100 Yr"
Flow (cfs)
200
150
100
50
0
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J251 Result:Outflow
Run:DBDPA 100 Yr Element:52 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 12F Result:Outflow


Junction "J260" Results for Run "DBDPA 100 Yr"
300
250
200
150
100
50
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J260 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 16F Result:Outflow

Junction "J262" Results for Run "DBDPA 100 Yr"




Junction "J261" Results for Run "DBDPA 100 Yr"
80
70
60
50
40
30
20
10
0
Flow (cfs)
00:00
03:00
06:00
09:00
12:00
15:00
18:00
21:00
00:00
01Jun2012
Run:DBDPA 100 Yr Element:J261 Result:Outflow
Run:DBDPA 100 Yr Element:BASIN 17F Result:Outflow

# APPENDIX D

PHOTOGRAPHS

***H1:*** *Hawthorne Ditch at Covington Street Discharge (irrigation on left, storm on right)*

***H2:*** *Hawthorne Ditch at Reed Court (inlet end is shown)*

***H3:*** *Hawthorne Ditch at Reed Court (outlet end is shown)*

***H4:*** *Hawthorne Ditch at Plateau Lane (inlet end is shown)*

***H5:*** *Hawthorne Ditch at Plateau Lane (outlet end is shown)*

***H6:*** *Hawthorne Ditch looking downstream at Plateau Lane*

***H7:*** *Hawthorne Ditch at Sprucewood Street (inlet end is shown)*

***H8:*** *Hawthorne Ditch at Sprucewood (outlet end is shown)*

***H9:*** *Hawthorne Ditch looking downstream at Sprucewood Street*

***H10:*** *Hawthorne Ditch at Reservoir Road (inlet end is shown)*

***H11:*** *Hawthorne Ditch at Reservoir Road (outlet end is shown)*

***H12:*** *Hawthorne Ditch looking upstream from Reservoir Road*

***H13:*** *Hawthorne Ditch Siphon Inlet and Waste Gate*
*(West of the Albert Street box culvert outlet)*

***H14:*** *Hawthorne Ditch Siphon Inlet (Top View of Siphon Trash Rack)*

***Element 1:*** *Low Flow Pipe Inlet located in Detention Pond 103*

***Element 1:****Inlet End of Pipe*

***Element 1:*** *Outlet End*

***Element 2:*** *Looking South from Pond 103*

***Element 2:*** *Looking South into Bottom of Pond 104*

***Element 3:*** *Inlet End of Pipe Covered with Debris, Located in Pond 104*

***Element 3:*** *Outlet End, Looking Downstream from Homestead Street*

***Element 4:*** *Looking South from Homestead Street,*
*Element 4 is the Bottom of Pond 104*

***Element 5:*** *Inlet End of Pipe in Pond 101 is Covered with Debris*

***Element 6:*** *Inlet End of Culverts Under South Pitch Drive*

***Element 6:*** *Looking Upstream From South Pitch Drive*

***Element 6:*** *Looking Downstream of South Pitch Drive*

***Element 6:*** *Looking North From Avenue A*

***Element 6:*** *Looking Downstream From Avenue A to Plateau Lane*

***Element 6:*** *Inlet End of Culverts Under Avenue A*

***Element 6:*** *Looking Upstream From Plateau Lane To Avenue A*

***Element 7:*** *Plateau Lane Box Culvert Inlet*

***Element 7:*** *Looking Downstream From Plateau Lane*

***Element 8:*** *Outlet End of Pipes Under Pedestrian Path*

***Element 8:*** *Looking North from Top of Pond 100, Pond Riser is in Foreground*

***Element 10:*** *Looking Upstream from Avenue A, Element 9 Outlet is Indicated.by Red Box*

***Element 10:*** *Culvert Inlet End at Avenue A*

***Element 10:*** *Looking Downstream from Avenue A*

***Element 11:*** *Plateau Lane Culvert Inlet End*

***Element 11:*** *Looking Upstream from Plateau Lane*

***Element 11:*** *Looking Downstream from Plateau Lane*

***Element 12:*** *Looking Upstream from Twilight Drive, Also Shows Element 7 Channel Entering on Right & Element 11 Channel Beyond*

***Element 13:*** *Twilight Drive Box Culvert Inlet*

***Element 13:*** *Twilight Drive Box Culvert Outlet*

***Element 14:*** *Looking South from Twilight Drive Box Culvert*

***Element 14:*** *Looking Upstream from Leroy Street*

***Element 14:*** *Leroy Street Looking Upstream*

***Element 14:*** *South of Roberts Court Looking Upstream*

***Element 15:*** *Albert Lane Box Culvert Inlet*

***Element 16:*** *Downstream of Albert Lane Looking South*

***Element 17:*** *Looking North Along Reservoir Road*

***Element 17:*** *Looking Upstream From Near Reservoir Road*

***Element 17:*** *Reservoir Road Box Culvert Inlet*

***Element 17A:*** *Looking Downstream from Reservoir Road*

***Element 17A:*** *Looking North from Longview Road*

***Element 17A:*** *Longview Road Box Culvert Inlet*

***Element 18:*** *Looking Downstream from Longview Road*

***Element 19&20:*** *Looking Upstream from Highway 44 Box Culvert*


***Element 20:*** *Highway 44 Box Culvert Inlet*

***Element 20:*** *Outlet of Highway 44 Box Culvert*

***Element 20&21:*** *Looking Downstream from Highway 44 Box Culvert*

***Element 22:*** *Looking North from Green Valley Drive*

***Element 22:*** *Looking South from Green Valley Drive*

***Element 22:*** *Green Valley Drive Box Culvert Inlet*

***Detention Pond 100***

***Detention Pond 100:*** *Riser*

***Detention Pond 101:*** *Looking North from Dam*

***Detention Pond 101:*** *Looking West, Note Outlet Pipe covered with Debris*

***Detention Pond 102:*** *Inside of Riser*

***Detention Pond 102:** Looking North from Dam,*
*Riser in Foreground*

***Detention Pond 103:** Looking North from Top of Dam,*
*Riser in Foreground*

***Detention Pond 103:*** *Secondary Pipe Inlet*

***Detention Pond 103:*** *At Spillway looking North*

***Detention Pond 104:*** *Inlet End of Pipe Covered with Debris*

***Detention Pond 105:*** *Looking South from Longview Road*